The chute carrying the bull chain and the gravity chute are both to be built of reinforced concrete. The chain chute is to be protected by replaceable 2½-in. half-round steel bars; the gravity chute has its wearing surface protected by embedded 60-lb. rails.

The "Proposed Power House," shown in Fig. 1, is not to be built at this time, but its location is shown in these drawings as a matter of convenience.

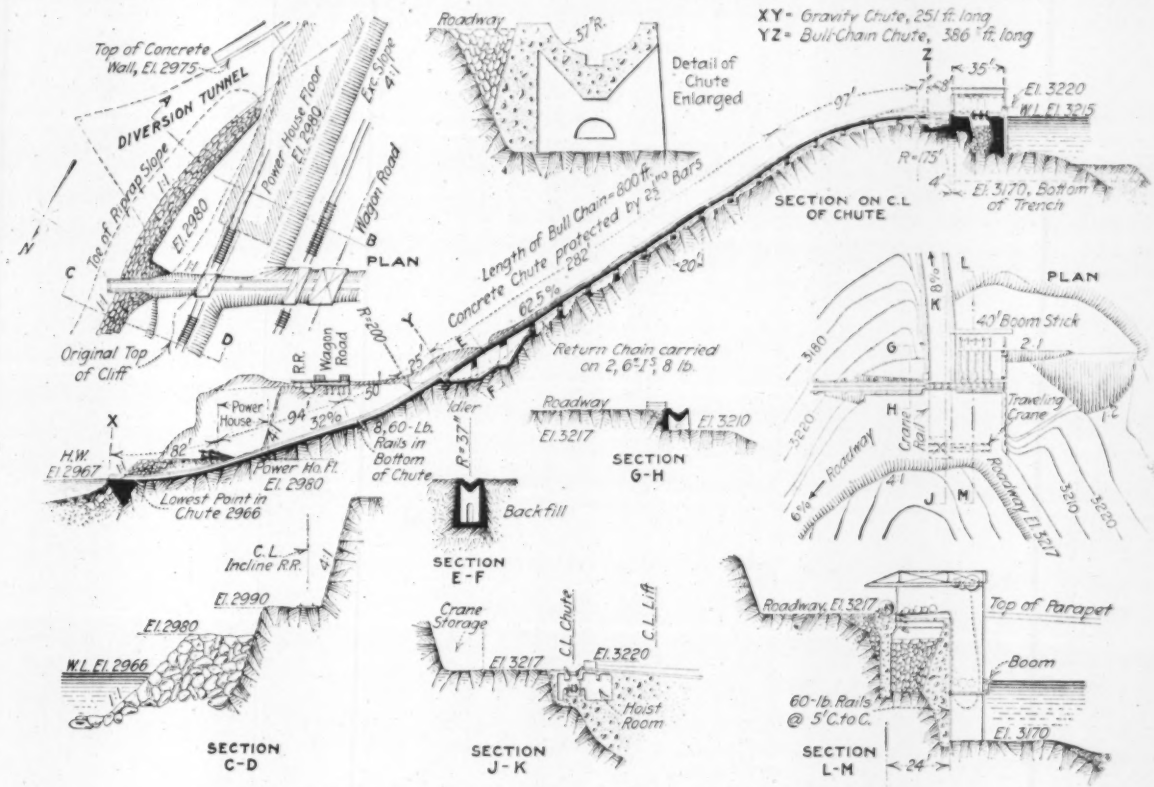

FIG. 1. DETAILS OF LOG-HANDLING EQUIPMENT AT ARROWROCK DAM

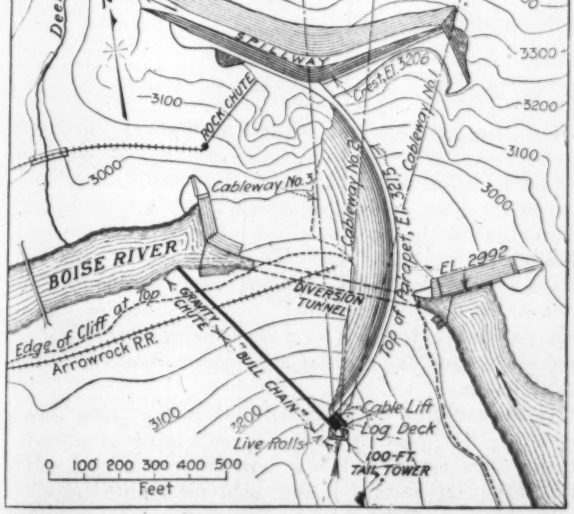

FIG. 2. LOCATION OF LOG HANDLERS AT ARROWROCK DAM, BOISE, IDAHO

This log conveyor will be one of the final construction features in connection with the Arrowrock Dam which is being built on the Boise River about 20 mi. above the city of Boise, Idaho, to provide storage for use on the Boise project of the United States Reclamation Service. Other features of the work have been described in *Engineering News* of June 6, 1912, Jan. 16, Mar. 20, and July 17, 1913; July 11, 1914 and Feb. 25, 1915. Water is now being stored in the reservoir and will be used for irrigation this season. The work is located in the Central Division, of which F. E. Weymouth is Supervising Engineer. The writer is Construction Engineer, in direct charge of the work at Arrowrock. James Munn is Superintendent of Construction. The log-handling equipment was designed by Walker R. Young, Assistant Engineer.

The Use of Drainage Water from Irrigation Tracts for further irrigation is suggested by R. A. Hart, of the United States Reclamation Service, in a recent paper presented before the Utah Society of Engineers. In one locality a drainage system which was installed at a cost of $5000 developed a flow of 1 cu.ft. per sec., a sufficient amount to irrigate 120 acres in that locality. As water rights in that section are worth $50 per acre, the water developed by this drainage system has a value of $6000. Small drainage systems often develop large quantities of water. In one case a drain only 1000 ft. in length, put in at a cost of $200, developed a maximum discharge of 1¼ cu.ft. per sec. during the irrigation season. On a 30-acre tract near Ogden, Utah, which is fully underdrained, the drainage water is led to a sump, from which it is pumped by a centrifugal pump, an electrical motor-driven unit, to a reservoir at the highest point of the tract. In this way the entire tract of 30 acres is irrigated with its own drainage water, having no outside source of supply. It will be understood, of course, that in this case the water stored in the underground strata merely acts as a reservoir to store up the surplus rainfall.

Hydraulic Redevelopment at Turners Falls, Mass.

By Howard M. Turner*

To follow the steps in the interesting redevelopment at Turners Falls it is necessary to know the history of the local power company. In 1792 the company was chartered as the "Proprietors of the Upper Locks and Canals of the Connecticut River." A dam was built at about the site of the present one, and a canal for navigation, traces of which still show, was dug from the dam to Montague City, 2½ mi. down the river. In 1866, after the railroads had rendered navigation on the Connecticut River unnecessary, the Turners Falls Co. acquired the property. A new timber-crib dam, still in place, was built, and a canal 50 ft. wide and 10 ft. deep develop it for the growing business. The scheme finally adopted consists of widening and deepening the present canal, extending it to Montague City and building a new power station there to develop the whole power.

From the headgates and through the town of Turners Falls, where the canal is in rock, the section is 127 ft. wide and 18 ft. deep, with almost vertical sides for a length of 3900 ft. About 500 ft. of this is new canal; over this section is a skew railroad bridge and under it a sewer crossing consisting of an inverted double siphon with a dip of about 16 ft. Gates at each end enable either tube to be cut off and cleaned out.

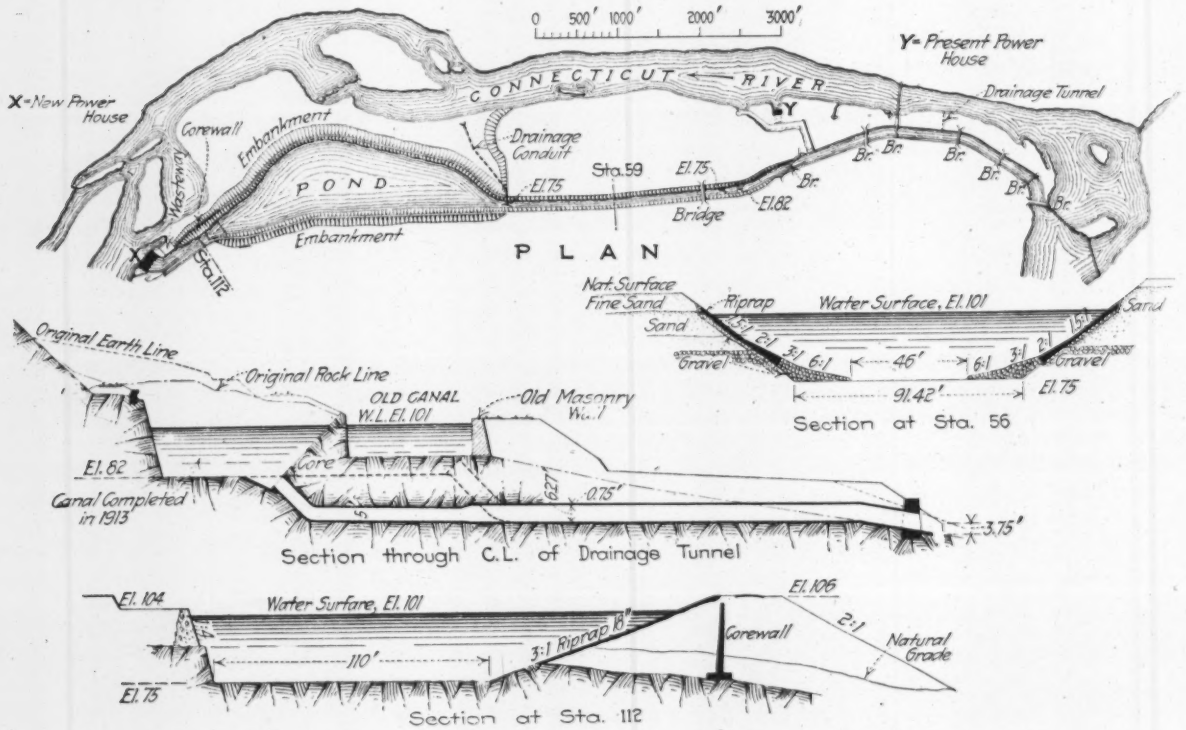

FIG. 1. GENERAL LAYOUT AND CANAL SECTIONS; TURNERS FALLS REDEVELOPMENT

was dug for power purposes. Water was leased to mills constructed between it and the river, the power being generated under heads varying from 33 to 39 ft.

In 1904, with the development of electric-power transmission, the canal was extended; a hydro-electric power station was built with provision for 7000 hp. of equipment and utilizing a fall of 39 ft. The canal at the headgate end was widened to about 130 ft. for a distance of about 800 ft. downstream, and eight new headgates were put in on the shore side of the old ones.

New Power Works

Below Turners Falls the river drops about 20 ft. in the 2½ mi. to Montague City. The company owned this additional power, and plans were made in 1910 to

The canal broadens out, beyond the town, in an earth cut to a section 25 ft. deep and 86 ft. wide on the bottom, with sides sloped 1½ on 1, and lined. This section extends for a distance of 3300 ft. At this point, where the natural surface drops off to a general level of about 10 ft. below water line, the canal widens out into a forebay pond of an average width of 600 ft. and a length of 3000 ft., with earth embankments on either side, narrowing up again to a canal section 150 ft. wide and 25 ft. deep for a distance of about 1000 ft. to the power house.

At the entrance to the pond, before the grade rises, there is a 5-ft. conduit to the river, serving as a drain during construction and as a permanent means of emptying this deep section of the canal for maintenance.

The bottom of the pond is slightly irregular, and to get the required effective cross-sectional area it was nec-

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essary to dig a channel 300 ft. wide to a depth of 10 ft. below water line, through a rise of ground in the middle. The pond has an area of about 60 acres, giving storage enough to take care of sudden increases of load at the power house without seriously drawing down the head before the canal headgates can be adjusted to the increased draft through the canal.

The embankments of the pond, which are carried 4 ft. above the maximum pond level, vary in height from 9 to 21 ft. Except for a small length at the lower end, they are built without core wall, from the material excavated from the main canal. Specifications call for a portion of the embankment on the inside below the assumed hydraulic grade line to be of fine sand or sandy clay, spread in 6-in. layers, watered and rolled once, the unrolled portion to consist of sand or sand and gravel. The canal banks are riprapped with 18 in. of stone on the inside. At the lower end, where the pond narrows up to the lower canal, there is a natural bank on one side and an embankment with a concrete core wall to rock on the river side. The core wall is set on a footing 10 ft. wide on the rock, in order to give a large area of contact.

Upstream from the end of the core wall, where the embankment is still near the river and the subsoil above the rock, which is about 20 ft. below the surface, is coarse sand, a cutoff of loam fill 15 ft. wide was put in for a distance of 300 ft. to prevent seepage.

Just upstream from the power house on the river side of the canal is the wasteway with a concrete spillway and ten wooden gates, 12 ft. wide by 10 ft. high, between piers. These gates are operated by hoists on a concrete floor across the piers, each hoist being geared to a shaft which is driven by a motor controlled from the power-house switchboard. The wasteway will pass full canal flow and will be used as a spillway in case of a sudden dropping of load, to remove the ice from the canal and to aid in emptying the waterway. From the end of the wasteway the river wall of the canal makes a curve, joining the end of the head wall of the power house, which is set with its long axis nearly parallel with the river. This wall, containing the racks and headgates, is at an angle of 20 with the center line of the canal. The racks are built of special rack bar spaced 3½ in. clear and with the bars turned slightly upstream. Behind the racks are piers separating the intake into penstock chambers; these in turn are divided into three parts by intermediate piers.

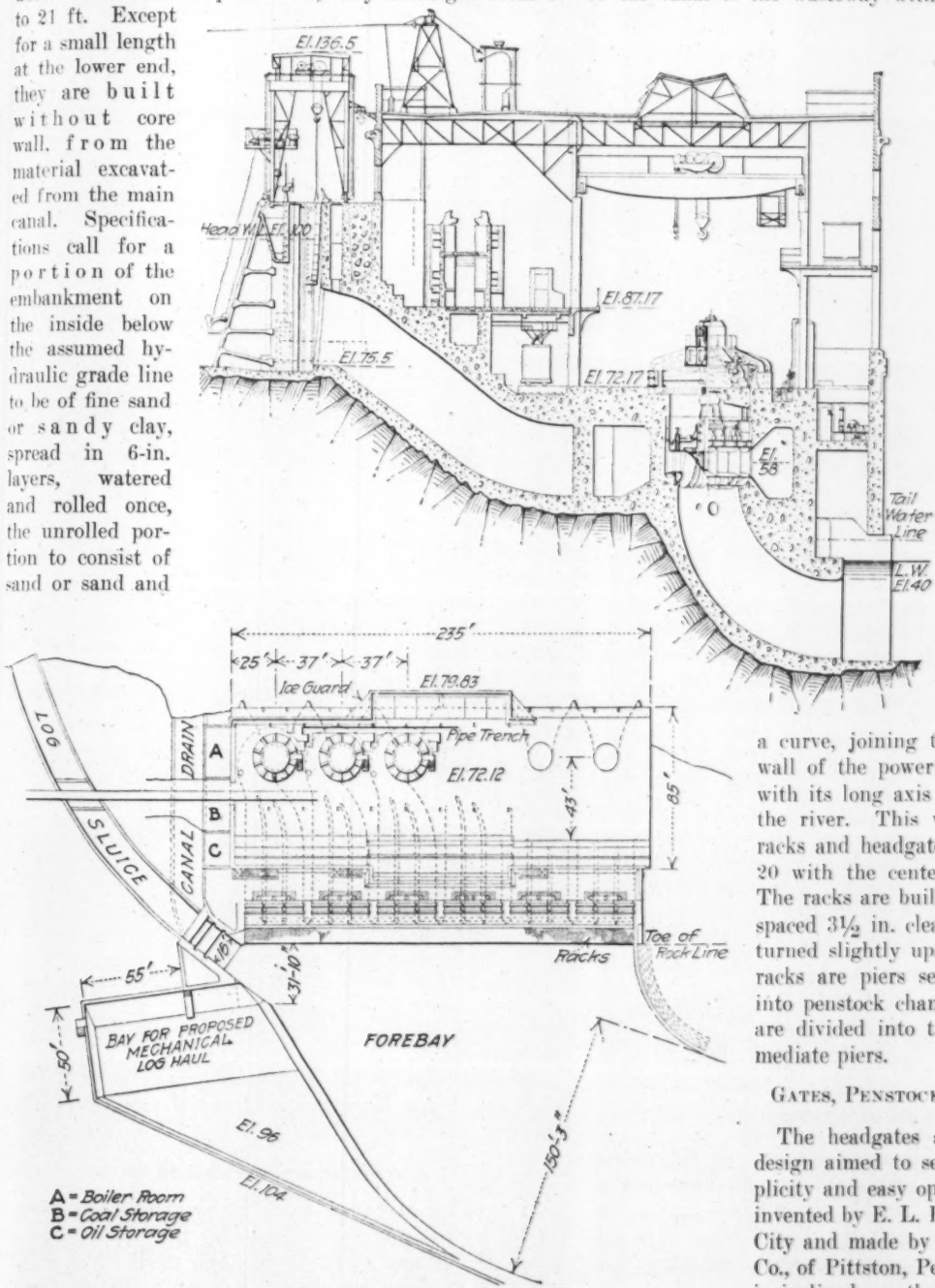


FIG. 2. PLAN AND ELEVATION OF POWER HOUSE

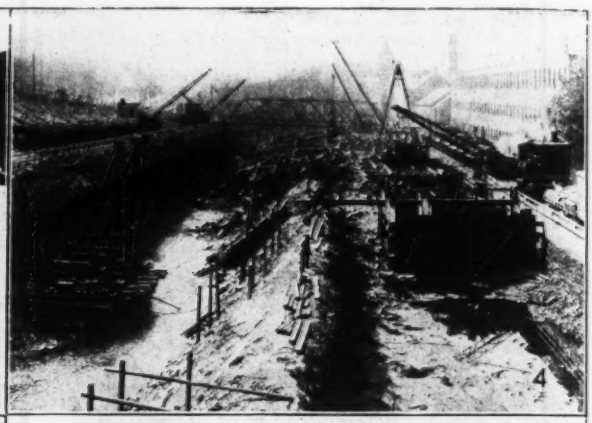
Gates, Penstocks and Log Sluice

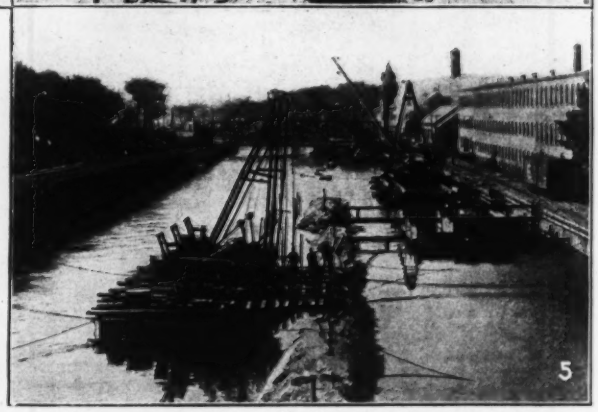
The headgates show an interesting design aimed to secure tightness, simplicity and easy operation. They were invented by E. L. Broome of New York City and made by the Exeter Machine Co., of Pittston, Penn. The deck plate is inclined, so that the forces on the gate (weight plus the vertical component of the water pressure) keep the supporting cables always in tension and no rigid stems are required. The gates run on a continuous chain of rollers between tracks on the gates and guides. An electric gantry crane running on the head wall will raise these gates; and also

a mechanical rack-raking device will be carried by it.

The penstocks, of reinforced concrete, curve down from the headgates to the scroll cases of the wheels, which are set 18 ft. above low water and discharge out into the river. The penstocks, scroll cases and draft tubes all have a facing on the inside, 1 ft. thick, of 1:2:4 broken-stone concrete, poured at the same time as the surrounding concrete, which is a mixture equivalent to 1:3:5, with gravel dredged from the tailrace in the river as aggregate. Large stones are embedded in the gravel concrete when possible.

The power-house design is shown in Fig. 2. The superstructure is of brick and steel, with concrete floors, and a concrete roof supported on trusses. At the downstream end is a submerged gate, 16 ft. wide by 8 ft. high, leading to the river for emptying the canal, and above this a sluice 16 ft. wide for handling the log drive which passes Turners Falls each summer and which it is planned to take through the new canal. The logs are carried through the canal to avoid the necessity of letting enough water down the river to carry the drive through as in previous years—enough water being required to rob the power plant of about 1000-hp. capacity. Moreover, the proposed mechanical log haul shown in Fig. 2 has been provided for to further reduce the loss of water. By the expenditure of some 50 hp. in electric current the logs can be taken out of the canal and slid into the river with only enough water to "grease the ways."





FIGS. 3, 4 AND 5. ENLARGING THE UPPER CANAL AT TURNERS FALLS

Fig. 3—New canal dry, old canal full; X is entrance to tunnel. Fig. 4—Water out of old canal on Sundays; coffer-dam in old canal for constructing tunnel entrance. Fig. 5—Drilling and removing core

Generating Station

The normal head to be made available for the wheels is to be 55 ft. with full load on all units. The final development will provide six 6000-kw. units, the present installation being three. Foundations were put down for all six and the power-house superstructure was completed for four.

The wheels are of a vertical-shaft single-runner type, 9½ ft. in diameter and rated at 9700 hp. with full gate under 54 ft. head and a speed of 97.3 r.p.m. The governors, designed with the turbines, use water pressure from a central pumping outfit with motor-driven centrifugals and sump tanks in a pit in the center of the station over the tailrace. In the governor-pump pit are the sump tanks and pump for the central gravity oiling system; the main tank and filter are on a gallery above, from which the oil flows to the guide and step bearings of each unit. Each generator is a 7500-kv.-a. three-phase 60-cycle 6600-volt revolving-field alternator. Upon the generator frame rests the bridge which supports the Kingsbury thrust bearing; above this is the exciter.

The transformers are on the main floor above the penstocks and will comprise four 66,000-volt banks for the out-going long-distance lines and two 13,200-volt banks for local transmission. The switchboard is located above the 13,200-volt transformers in a room that is 15 ft. above the main generator room, and has the 6600- and 13,200-volt bus galleries immediately behind it and the high-tension rooms on either side above the 66,000-volt transformers.

This development has been made gradually, each forward step being taken as the financial position of the company warranted and the growth of business required.

Construction of Canals

In 1912 the first work along the plans mentioned was started by enlarging and deepening the upper end of the canal. A new canal, 18 ft. deep, about 60 ft. wide and 2100 ft. long, was started parallel to the old one, leaving between the two a "core" of rock. The necessary relocations of railroads and highways were made, and the old bridges were replaced by bridges spanning the two canals. By coffer-dams at each end of the cut, the work was done in the dry and the original canal kept in operation during the construction. To drain

the cut during construction and also to serve as a permanent drain for the deeper part of the canal until such time as the "core" was removed, a tunnel was dug from the new canal under the old one to the river.

Owing to the small size of the upper canal section, smoothness of the sides and bottom was important. The specification called for the rock excavation on the bottom and finished side of the new canal to be carried at all points to the given line, but at no point more than 6 in. back of the line. While in some places this was not possible, on account of the nature of the rock, the result obtained was very good on the whole. This trimming was done with small drills and by hand after the main excavation had been done. Work was finished here in July, 1913.

Construction of Dams

An examination of the old timber-crib dam, built in 1866 and divided in the middle by an island, showed shutdown of the electric power, a small waterwheel was put in the abutment, with its gates opened from the headgate house.

The old timber-crib dam, with high and substantial flashboards erected on it, served as a coffer-dam on the upstream side of the river construction. But leakage through this old crib and the backwater from the river below required some further coffer-damming, particularly at a deep point in the river just below the sluice gates. The work on the new headgates required coffer-dams on each side—one in the river and one in the canal—as the canal was kept in operation continuously during the progress of the work.

In order to pass the water to the new sluice gates under the dam, it was necessary to remove a portion of the old timber dam and dredge out a considerable amount of filling that had been put on this upstream side. This was done by derricks on the new concrete dam, and a scow with a derrick, operating orangepeel buckets; divers were

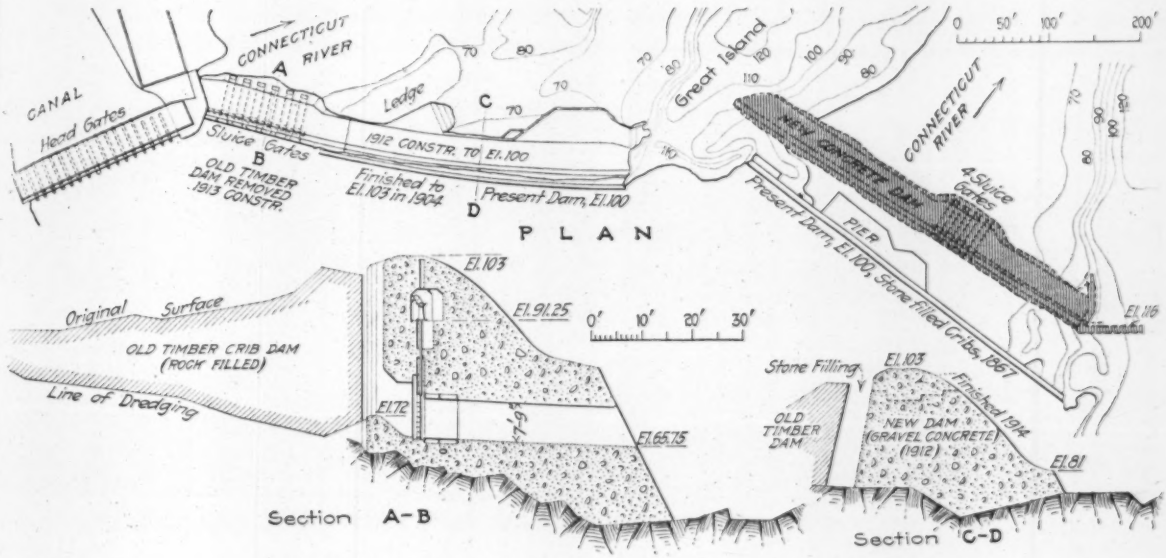


FIG. 6. PLANS AND SECTIONS OF DAMS AT TURNERS FALLS

that the Turners Falls side needed repair. It was decided to reinforce part of this by a concrete dam built just downstream from it, which could be part of the permanent structure from the island to the shore. Accordingly, in the summer of 1912, 350 ft. of concrete dam was built, the space between it and the old timber dam being filled with stone. This concrete dam was not carried up to the final height, as the rights to do this were not at that time secured. Keys were left in the top of the concrete dam, which was planked over back to the old dam.

In 1913 the remainder of the dam on the Turners Falls side of the island was built. This section, 150 ft. long, has six submerged gates 8 ft. wide by 7 ft. 9 in. high, operated from a tunnel in the dam. With this section of dam was built a new abutment on the Turners Falls side, and new canal headgates. The latter were put in at the location of the original headgates and extended from the abutment to the river end of the headgates built in 1904. They are nine in number, 8½ ft. wide by 12 ft. high, motor-operated by geared hoists inside a headgate building. To serve as a relay in case of any employed wherever necessary to chain out old logs. The upper part of the dam was loosened by small charges of dynamite.

Power-House Construction

In the spring of 1914 work was started on the extension of the canal, the construction of the new power station at Montague City, and the removal of the rock "core" in the present canal.

In the river about 20 ft. from the power house and running parallel to it, from the upstream end nearly to the downstream end, is a natural ridge of rock that is out of the water at low stages of the river. During low water in the summer of 1913, sills were pinned to this rock wall and concreted. On these sills a part of the contractor's coffer-dam of rock-filled timber cribs was placed. The space between the ridge of rock and the shore downstream from the power house was dammed with a double row of steel sheeting driven to clay beneath the level of the bottom of the tailrace.

The contract for the power-house substructure and the canal excavation immediately behind the power house was

let to Fred. T. Ley & Co., Inc., of Springfield, and work was started early last spring. This is nearly completed.

The wooden forms for the draft tubes were built complete on the bank, then sawed into four parts for handling and reassembled in place. The forms of the penstocks and scroll cases were built up in place. Concrete was mixed in two mixers, one for the 1:2:4 used in lining the waterways and one for the 1:3:5 mass concrete. The former mixture used sand and crushed trap brought in on cars and was deposited by a tower and chutes; the latter used river gravel and was dumped in through short chutes from cars on a track. The mixtures were kept separate by sheet-iron baffles, which were drawn up as the concrete was placed against them.

Forebay Work

The contracts for the canal dug in 1912 and for both sections of the dam and the headgates were let to the Bates & Rogers Construction Co., of Chicago and Spokane. This work is now entirely completed. The contract for the extension of the canal was let to Holbrook Cabot & Rollins Corporation, of New York and Boston, and work was begun early in the spring of 1914. The excavation of the canal above and below the pond is being done with steam shovels and standard-gage locomotives, with 12-yd. cars. The fine material, most of which is in the cut above the pond, is being used for the rolled embankment and the coarser material, for the unrolled portion of the embankment. The remainder of the material is dumped partly along the river bank near the power house and partly to fill some lower land outside the canal and embankments.

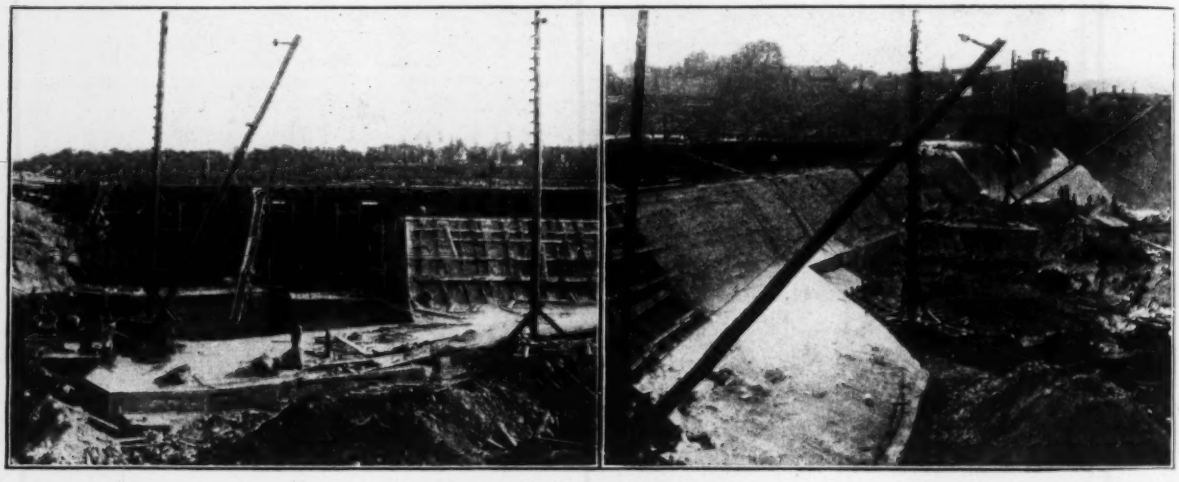
To cut off the water from the end of the present canal, a line of steel sheeting was driven to rock, extending the full width of the canal, behind which the material was excavated in the dry. The walls at this point were put in, in sheeted trenches. After these walls were complete, the main-canal excavation at this point was done, and a temporary wooden bulkhead with gates in it, to allow for filling the canal, was constructed from wall to wall; this will be left in until the canal has been tested out.

The construction of the drainage conduit, which was completed last November, enables the contractor to handle the water encountered in the cut without pumping. The original ground-water level in this cut was about 15 ft. above the grade of the bottom of the canal, but it is being lowered as the excavation progresses.

The pond embankments are being constructed as follows: Two tracks are laid the length of the embankments at such a location that the material for the rolled embankment and the ordinary embankment may be separated. The trains from the steam shovel are dumped on the embankment, spread to the specified thickness by a Jordan spreader, watered with a tank car behind the spreader and rolled with a four-horse grooved roller. After the embankment is up to the level of the tracks, the latter are raised and a new layer or layers deposited, the contractor not being required to roll the embankment under the track. After the embankment is complete, the riprap is hauled out on it, dumped from the track and laid in place.

Engineers

The engineering work in connection with the detail plans and construction is being done by the Turners Falls company's engineers, with the writer as Hydraulic Engineer in charge, and H. A. Moody in charge of the power-house work. F. L. Hunt is the Chief Engineer of the system. A. T. Safford, of Lowell, is the general consulting engineer, and F. P. Stearns of Boston has been consulted in regard to embankments and cutoffs. The turbines and governors were designed and built by the I. P. Morris Co., of Philadelphia; the electrical equipment was furnished by the General Electric Co., of Schenectady.



FIGS. 7 AND 8. BUILDING CONCRETE DAM BEHIND OLD ROCK-FILLED TIMBER CRIB

A Long Fishing Pier extending from a point 1000 ft. into the ocean across the beach at Lonelyville, on Fire Island, and 500 ft. into Great South Bay is being built for the Fire Island (L. I.) Fish Co. The construction of the pier was undertaken to afford easy transportation of fish from the ocean pounds and to serve as an attraction to the many amateur fishermen who visit this vicinity from neighboring towns. At the ocean end the pier is 90 ft. wide. The land section is 14 ft. wide. It is stated that about 800 piles will be required of lengths varying from 45 to 75 ft. In the ocean portion the piles are set 10 ft. apart. In the 10-ft. intervals there will be stretched a series of nets which will lead up to a large trap at the end of the pier. The pier is of the same type as the one at Belmar, N. J. The pier on the bay side connects with a canal 1000 ft. long, which is being dug in the shallow water so as to accommodate deep-draft boats for carrying fish to the mainland. It is estimated that the project will cost between $60,000 and $75,000. It is expected to have the pier in operation by August.

New Type of Pick-Up Street Sweeper

One of the machines which attracted considerable attention at the exhibition of the street-cleaning apparatus in New York City last November (see *Engineering News*, Dec. 3, 1914) was the Merritt pick-up sweeper, shown in the accompanying illustration. This sweeper was invented by a New York City traffic policeman and was first tried in New York City in the summer of 1912. The machine was used by contract and the work paid for at the rate of 48c. per 1000 sq.yd. of street surface cleaned.

From July 25 to 31, 1912, inclusive the machine cleaned 288,914 sq.yd. of pavement, at rates ranging from 16,490 to 61,779 sq.yd. per working day. In August it cleaned 916,968 sq.yd. of pavement in 21 working days, or at an average rate of 43,600 sq.yd. per day. The amount of sweepings was 860 cans. The records show that in September 772,570 sq.yd. were cleaned in 23 working days, an average of 33,600 sq.yd. per day. The amount of sweepings, as in general throughout the record, averaged from 40 to 60 cans per day, or a can per 1000 sq.yd. In October the area cleaned was 914,617 sq.yd., or about 33,000 sq.yd. per day.

MERRITT PICK-UP STREET-CLEANING MACHINE ON NEW YORK CITY STREET

During 1913 two of these machines were used from June to November. The contract price for their work was 41c. per 1000 sq.yd. of street cleaned. The records of the two machines were much the same as that already given for the earlier machine in 1912. The best month's record was in October, 1913, when the two machines cleaned 3,109,394 sq.yd. in 31 working days, an average of 100,000 sq.yd. per day (50,000 sq.yd. to each machine). But this appears to have been a very extraordinary performance.

The work of both years included pavements of granite block, wood block and asphalt, but the pavements were chiefly asphalt, as nearly as can be judged by a cursory examination of the Street Cleaning Department's official records, from which all the foregoing information is taken. No contract was made for the use of these machines in 1914, although the street-cleaning officials have no fault to find with their efficiency. It was demonstrated at the exhibition of street-cleaning machinery, previously referred to, that these machines could be used very effectively to pick up refuse swept into windrows by handbroom men.

Description of Sweeping Machine

The Merritt street-sweeping machine has brooms on either side, revolving on axes at an angle to each other, and a rear broom to pick up the windrow made by the side brooms. The rear broom sweeps the material over a shoe or dustpan to a conveyor, which discharges through two chutes into cans on a revolving platform. The brooms and conveyor are driven through gears, which are in turn driven by chains from the rear wheels.

Both front and rear wheels run on roller bearings. The main frame of the wagon is of channel-bar steel and all castings are of cast steel. All the revolving parts are equipped with roller bearings. The side brooms are 28 in. in diameter and the rear broom is 38 in. A two-horse machine sweeps a 7-ft. strip and a three-horse machine a 10-ft. strip.

The shoe drags lightly on the pavement. It is made of spring steel, in flexible sections, allowing it to conform to depressions in the pavement. The conveyor is made in hinged sections of galvanized sheet-iron on sprocket chains, and is inclosed in a sheet-iron casing, which also forms the double chute for depositing the sweepings.

The can carrier is a revolving steel rack, made to hold eight cans. The rack is mounted on a specially constructed roller bearing. Two cans are filled at a time, and when filled are quickly shifted by the driver by means of a lever beside his seat and new cans spotted under the chutes. The cans are of the standard type.

The newest sweeping machine is provided with a 110-gal. galvanized-iron water tank arranged to sprinkle lightly the entire width of the sweeper. Variation in the flow is obtained by two perforated brass pipes, one giving a light flow, the other a slightly heavier flow. One or both pipes are used, according to the street.

The second machine embodying some of these improvements was built in 1912 and used on New York City streets, with the record given in this article. Since the New York City work was done the conveyor has been improved and the sprinkling device added. Two of these machines are in use in Yonkers, N. Y.

Designing a Steel Dome

By A. W. Earl and Thomas F. Chace*

Before designing the steel-frame dome of the Horticultural Palace at the Panama-Pacific International Exposition—described in *Engineering News* of Oct. 15, 1914, and July 15, 1915—a careful search was made of existing literature on framed-dome design. Discussions of the subject are conspicuous in American professional papers by their absence, and this is especially true with respect to methods of providing for wind and similar stresses produced by other than uniform vertical loads.

The size of this dome, coupled with a desire to keep the framework light and airy in conformity with the character of the building, made a careful study of the problem well worth while.

An outline diagram of the dome framing, with dimensions, is given in Fig. 1.

Available Methods of Design

There are two methods commonly employed in the design of framed domes. The first, and perhaps the most

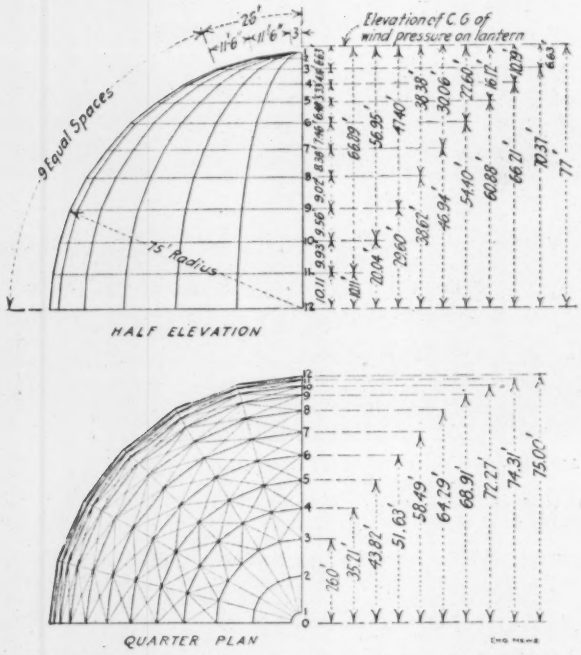


FIG. 1. DIMENSION DIAGRAM OF DOME, PALACE OF HORTICULTURE

common, consists of treating any two diametrically opposite ribs as forming a three-hinged arch, the hinges at the crown being replaced by a circular girder. In this method wind stresses are taken care of by considering the wind as a live-load on one side of the dome, applied only in the plane of the two ribs forming the arch. Diagonal rods are usually provided in the panels of alternate bays to give general stiffness to the structure.

The second method regards the ribs as members of a true dome wherein all primary stresses are resisted by compression in the ribs and compression or tension in the horizontal rings. Wind and all other than uniform vertical loads are taken care of by placing diagonals in every panel.

By applying this second method to structures of this type more material is saved than might at first be supposed. In the present instance the dome was completely designed by both methods, the second method resulting in a saving of one-third the steel called for by the first design in the portion of the structure above the plate-girder supports; that is, in the actual dome itself. An idea of the relative proportions of the respective frames may be gained from the statement that with the same number of ribs in both cases the ribs in the first design were 72 in. deep with chords of two 6x6x½-in. angles with 14x⅜-in. cover-plates, while the ribs as actually built are 36 in. deep with chords not exceeding two angles 4x4x$\frac{5}{16}$ in. with no cover-plates.

The writers lay no claims to originality in the following mathematical discussion, but rather have applied formulas and methods already developed in the design of similar structures. In the course of the discussion reference is made to books and papers on the subject by catchwords, while a complete list of the principal sources of information is given at the end of the article.

Rib and Ring Stresses

The rib and ring stresses were determined by the method outlined by E. Schmitt (Trans. Am. Soc. C. E.); no explanation is needed further than the load-and-stress summary and dead- and live-load diagrams shown in Fig. 2.

The ribs and rings of the dome of the Horticulture Building at the World's Columbian Exposition in Chicago were taken as a basis for a preliminary estimate of weight, and from this a tentative design was made. The dead-load of the roof covering was estimated by direct computation. Using the sections obtained from the stress sheet, a final estimate of weight was made, the stresses recomputed on the basis of the revised weights, and the members modified as necessary.

Diagonal Stresses

The problem presented in finding the diagonal stresses was slightly more intricate. Three methods were used, all based upon different assumptions. The first was as outlined in Hütte, the second somewhat similar to that given by Hazlehurst for stresses in a steel water tower, and the third like that used for the computation of stresses in a gas-holder guide-frame in an article in *Engineering News*.

First Method—The Hütte method is based (with a 50% correction) on simple resolution of forces at the several joints of a rib under the condition that one-half of the dome (on one side of the rib) carries full live-load, while the other half carries no live-load, which is assumed to be the worst condition of loading.

Let Q_1, Q_2, Q_3, etc., represent total dead- plus live-load per panel, beginning at the top; P_1, P_2, P_3, etc., dead-load per panel; A_1, A_2, A_3, etc., the angle between rib and horizontal plane; B_1, B_2, B_3, etc., the angle be-

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tween rib and diagonal measured in the roof plane; N_1, N_2, N_3, etc., the diagonal stress; then:

$$N_1 = \frac{Q_1 - P_1}{2 \sin A_1 \cos B_1}$$

$$N_2 = \frac{Q_1 + Q_2 - (P_1 + P_2)}{2 \sin A_2 \cos B_2}$$

$$N_3 = \frac{Q_1 + Q_2 + Q_3 - (P_1 + P_2 + P_3)}{2 \sin A_3 \cos B_3}$$

and so forth.

These formulas, with the exception of the figure 2 in the denominator, follow directly from the resolution of forces under the conditions stated. However, results obtained by Müller-Breslau and others from exact analyses gave stresses about one-half as large as those resulting from direct resolution of forces, and for this reason the 2 in the denominator is introduced.

The angles A and B are given in Fig. 3. They were used in the above formulas together with the loads noted in Fig. 2, which are the same as in Fig. 1. As there are no diagonals above ring III, Fig. 2, the diagonal between rings III and IV was called N_1, and the loads were summed down to this panel to get Q_1 and P_1.

It might be noted that the exact analysis of a dome by methods of least work, while presenting an interesting and instructive academic problem, is by far too laborious and complicated for commercial office use. On the other hand, the method above described is seen to be largely empirical; its application requires experience and judgment in the selection of live-load. In the present instance a live-load of 20 lb. per sq.ft. of horizontal projection was taken.

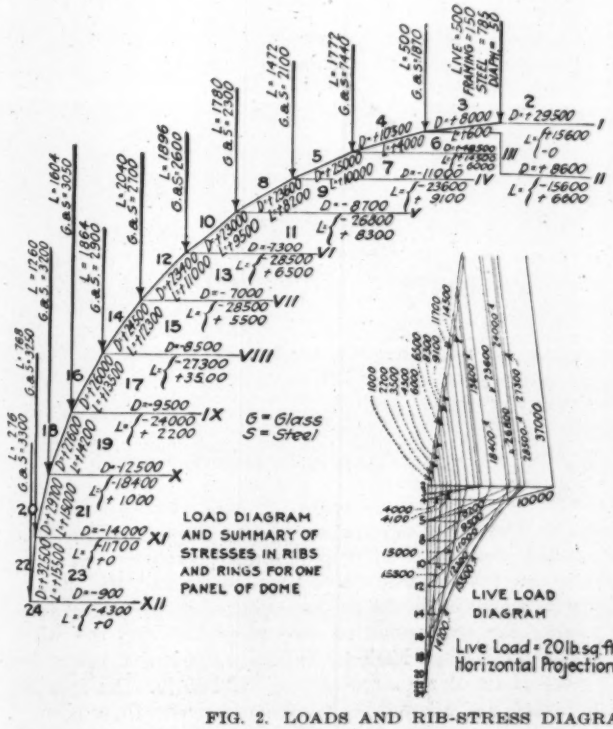
LOAD DIAGRAM AND SUMMARY OF STRESSES IN RIBS AND RINGS FOR ONE PANEL OF DOME
G = Glass
S = Steel
LIVE LOAD DIAGRAM
Live Load = 20 lb. sq. ft. Horizontal Projection
DEAD LOAD DIAGRAM

FIG. 2. LOADS AND RIB-STRESS DIAGRAMS

Second Method—On account of the empirical nature of the Hütte formula and the fact that it has regard to unilateral loading only but not to wind, it was thought advisable to check the results by a method based on wind pressure, which would be the most likely cause of stress in the diagonals of a glass-covered dome of this character. To this end the method for the solution of the stresses in a water tower to be found in Hazlehurst's treatise on the subject was adapted to the present case.

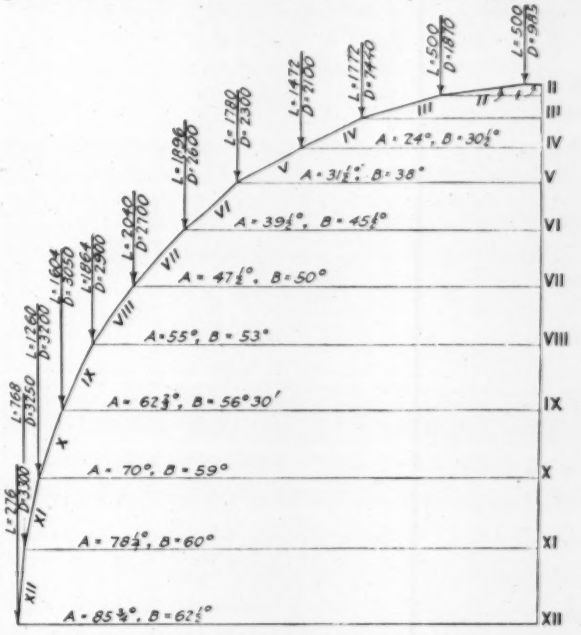

FIG. 3. ANGLES AND LOADS USED WITH HÜTTE METHOD

The method may be briefly illustrated by reference to Fig. 4, which represents a trussed pyramid with inclined posts, acted upon by wind. Let P = the total wind shear from all zones above; p = the total wind shear per rib, that is, P divided by the total number of ribs, or in this case $\frac{1}{6}P$. This means that the shear is assumed to be equally divided among the ribs. In addition to the forces p acting at each rib there will be a horizontal component of the rib stress. This component is most easily found by first computing the uplift at every point, assuming all ribs on the windward side of the neutral axis to be in tension while those on the leeward side are in compression, and further assuming that the magnitude of the tension or compression varies directly as the distance from the neutral axis. Knowing this vertical component and the angle of inclination of the ribs of the horizontal, the horizontal component of the rib stress may be computed. The resultant of the horizontal component of the rib stress and the force p at every rib is then found,

and finally this resultant is projected upon the two ring members. When this is done it will be seen that there is still an unbalanced condition at the point to which the diagonal furnishes the balancing component.

Third Method—A final method of computing the diagonal stresses was adapted from the method given in *Engineering News* for analysis of diagonal stresses in a gasholder guide frame. It may be seen from a study of the article that the bays most nearly parallel to the direction of the wind take the greatest diagonal stress, which in amount is equal to twice the average of all the bays. Thus, referring again to Fig. 4, the horizontal component of the maximum diagonal stress in any zone is

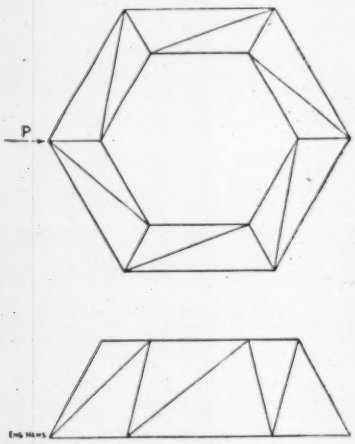

FIG. 4. DIAGRAM OF TRUSSED PYRAMID

equal to the total wind shear of all the zones above divided by one-half the number of panels. Having determined this horizontal component of the stress it is a matter of simple resolution of forces to obtain the total stress in the diagonal rod.

This method was followed in the analysis of the present dome, using graphical diagrams which are not of sufficient importance to reproduce.

Comparison of Results—The following table gives the summary of results obtained by the several methods:

Diagonal	Hütte Method	Hazlehurst Method	Eng. News Method
N_1	2,530	1,000	600
N_2	3,960	1,700	800
N_3	5,630	2,700	1,500
N_4	7,300	4,000	2,700
N_5	9,100	6,700	4,000
N_6	11,050	7,100	6,100
N_7	12,850	10,000	8,000
N_8	14,000	13,000	11,300
N_9	15,700	14,500	14,000
N_{10}*		36,000	36,000

*Diagonal in cylindrical part below spring line of dome.

Each diagonal was proportioned for the maximum of the three stresses given in this table. It will be noted that the three methods check fairly closely where the ribs of the dome approach the vertical, though they depart widely from each other where the ribs become more nearly horizontal.

It has been pointed out above that the Hütte method for finding the stresses in the diagonals is largely empirical. The same criticism may be made of the other two methods, since in each of these the assumption is made that the wind shear is distributed equally among the ribs, an assumption whose truth depends largely upon the stiffness of the horizontal rings. The close agreement shown by the preceding table may not always be realized.

It is therefore obvious that in the design of a structure of this nature a satisfactory design depends to a great extent on the judgment and experience of the designer, and while the methods here outlined are applicable to the general case of domes, no conclusion based upon this study of one dome should be applied to any other dome without careful consideration of the particular case.

Type of Rib and Ring Construction Used

After the stresses were computed careful consideration was given various types of rib and ring. The trussed type was decided upon because of its stiffness and its ability to resist secondary stresses due to bending. This latter consideration is important because the members are curved while the direct stresses between panel-points act in a straight line. The drawings of the dome given in a prior article, *Engineering News*, July 15, 1915, p. 112, show the type and makeup of sections chosen for the ribs and rings.

Where there is a heavy lantern on the dome, as in the present case, there should be a stiff ring at the top to insure a uniform distribution of this load.

Design of Supporting Tower

For the design of the dome the wind pressure was assumed at 20 lb. per sq.ft. of vertical projection, which amounts to slightly more than 30 lb. per sq.ft. on a plane normal to the wind with 40% reduction for a cylindrical surface. In designing the substructure or supporting tower, however, it was considered improbable that a wind pressure of 20 lb. would exist over the entire area of dome surface at the same instant. It was thought consistent to reduce the wind another 40%; if the first reduction took account of the circular horizontal section the second reduction takes account of the circular vertical section. The resulting pressure of 10.8 lb. per sq.ft. of projected area was used in designing the substructure. At the same time, the substructure itself and the lantern were loaded with 20 lb. per sq.ft. of projected area because the ornamentation formed pockets for the wind. The total wind load (including 120,000 lb. below the base of the dome structure) is 312,500 lb. An increase of 25% was allowed for wind stresses over those allowed for dead-load stresses, and therefore the equivalent horizontal dead-load was 312,500 × 0.80 = 250,000 lb. Fig. 5 is a plan diagram of the tower.

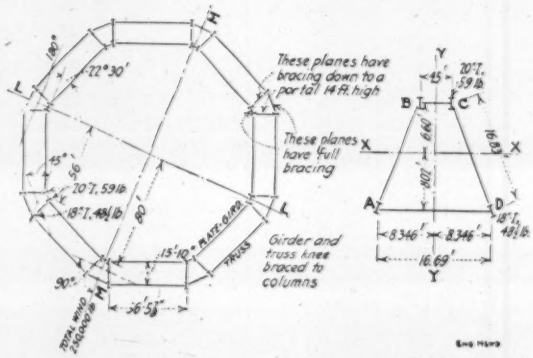
These planes have bracing down to a portal 14 ft. high
These planes have full bracing
Girder and truss knee braced to columns
72°30'
20°7, 59 lb
18°7, 49 lb
8.346'
16.69'

FIG. 5. PLAN DIAGRAM OF DOME-SUPPORTING TOWER AND PLAN OF ONE PIER

Two assumptions were made as to the distribution of the horizontal wind load among the eight piers—(1)

that the piers were loaded in proportion to their respective rigidities and (2) that they received equal loads. The most unfavorable conditions given by the two assumptions were used in the design. In calculating the stresses in the two nonparallel sides of a pier it was considered that each pier acted independently of the others, as a cantilever; but in calculating the stresses in the knee-braces of the truss and girder system and the stresses in the two parallel sides of the piers it was considered that each pier acted as part of a rigid structure with its neutral axis passing through the center of the tower.

First Assumption—The assumption that the piers take wind load in proportion to their respective rigidities may be stated as follows: The horizontal load at the top of each pier will be to the total horizontal load as the moment of inertia of each pier about an axis through its center of gravity normal to the direction of the wind is to the sum of these moments of inertia.

It was found by trial that for the outer columns 18-in. 48½-lb. I-beams would answer and for the inner columns 20-in. 59-lb. I-beams. With these sections and the geometrical shape of the pier known, the direction

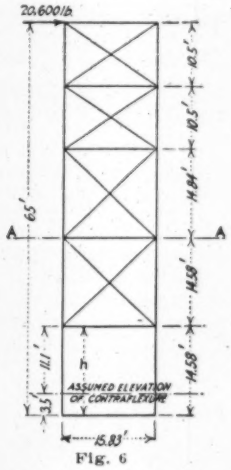

Fig. 6

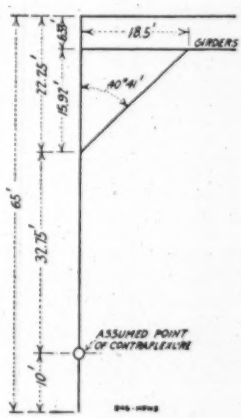

Fig. 7

of the wind was taken parallel to the long diameter of the tower octagon (this direction giving maximum stresses) and for each of the eight piers the moment of inertia about an axis at right angles to this direction of wind was computed.[1]

The wind load was distributed to the piers in accordance with the resulting values of moment of inertia. Then the wind load of each pier was resolved in the direction of the two principal axes of the pier; the pier was then to be designed to be strong enough in each of these two directions to resist the maximum wind component acting in that direction.

The maximum component in the direction of the longer axis of the pier was found at windward and leeward piers, being the full wind load coming on these piers, 38,000 lb. each. In the transverse direction (around axis *Y-Y*, pier plan in Fig. 5), the maximum component was the full wind load on the pier, 90° from this position, 24,400 lb.

Second Assumption—On the basis of equal wind loads for all the piers the force per pier was computed as 31,300 lb. equally on the pier whose long axis is in line with the wind and the pier at 90° from this point.

Combining the two assumptions, the maximum loads in the two directions of the pier are: 38,000 lb. in the direction of the long axis (from first assumption); 31,300 in the direction of the shorter axis (from second assumption).

Stresses in the Pier Columns

Portal Stresses—In the portals in the two nonparallel sides of the pier (*AB* and *CD* in Fig. 5), it was assumed that the columns would take horizontal shear in proportion to their respective rigidities. In the final design the inner columns were reinforced by 8x1-in. flange plates and the outer columns by 8x⅜-in. flange plates in the region of the upper part of the portal. Taking the respective moments of inertia of the full section of I-beam and flange plates the wind shear was divided between the two columns in the ratio of 2525 (Col. B) to 1304 (Col. A), the respective total moments of inertia about an axis normal to the slant side of the pier.

In the portal it was considered that the columns would be partially fixed at the base; the plane of contraflexure was assumed at 3.5 ft. from the bottom, which is approximately one-quarter the height of the portal (14.58 ft.) On this assumption the wind shear of 13,600 lb. on the inner column (Col. B) produces a bending moment at the top of the portal of 1,810,000 in.-lb., or a fiber stress (considering the angle between plane of portal and plane of I-beam web) of 8400 lb. per sq.in. The dead-load on one inside column amounts to 110,000 lb., or 3300 lb. per sq.in.; the compressive wind load due to overturning of the pier is 80,000 lb., reduced by the dome-rib uplift of 9300 lb., producing a net fiber-stress of 2100 lb. per sq.in. Thus the total extreme-fiber stress in the column at the top of the portal is 13,800 lb. per sq.in. Since this maximum stress occurs at the point where the column is supported laterally by the lower strut of the portal it is well inside safe limits.

For the outer column (Col. A), the wind shear of 7000 lb. produces a bending moment at the top of the portal of 930,000 in.-lb., or an extreme-fiber stress of 6700 lb. per sq.in. The dead-load of 21,500 lb. and the overturning compression of 80,000 produce unit compressions of 1060 and 3950 lb. per sq.in. respectively. Thus the total fiber stress in the outer column at the top of the portal is 11,710 lb., which, occurring at a supported point, is well on the side of safety.

Stresses in Parallel Faces of Piers—For the wind action in the inner and outer faces of the pier (*BC* and *AD*, Fig. 5), the following arbitrary assumptions were made: First, that the total load of 31,300 lb. wind shear is taken 75% by the bracing between the inside columns and 25% by the bracing between the outside columns; second, that there would be sufficient fixing effect at the base of the pier to cause contraflexure 10 ft. from the bottom of the pier when the tower is considered as a rigid whole tending to overturn about an axis through the centers of the bases of two opposite piers.

The total overturning moment on the tower was 26,525,000 ft.-lb. The vertical reactions on the piers caused by this overturning were considered as acting on the large interior columns only. Considering the reaction to vary with the distance of the column from the neutral axis of the tower, the maximum overturning load was found to be 84,000 lb. for the two columns of windward or leeward pier. However, this compression due to overturn-

[1]The moments of inertia about the two principal axes of the pier being first computed, the moments about any other axis making an angle a with one of the principal axes, say the Y-axis, was found by the formula $I = I_y \cos^2 a + I_x \sin^2 a$.

ing was not a governing stress in the design, as it was exceeded by the bending stress in the columns of the pier on the neutral axis of the tower, where the maximum wind shear occurs. On the two inner columns the shear is 75% of 31,300 lb., or 23,500 lb. Choosing for the section of the bent *BC* two 20-in. 59-lb. I-beams spaced 4½ ft. on centers, with one 16x⅜-in. plate on the outside of each web, and with point of contraflexure 10 ft. above the bottom, the maximum bending stress at the foot of the kneebrace was found to be 8300 lb. This combined with a dead-load stress of 4700 lb. per sq.in., but without compression stress from overturning of the tower (since this particular pier is at the neutral axis of the tower) gave a total extreme-fiber stress at the foot of the kneebrace of 13,000 lb. per sq.in. This is somewhat less than the stress of 13,800 lb. for extreme wind conditions in the inner columns due to portal action in the longitudinal face of the pier (that is, when the wind is at right angles to the direction just considered).

The latticing between the 20-in. I-beams was designed for a shear of 23,500 lb. below the kneebrace and (see Fig. 7) $\frac{23{,}500 \times 32.75}{15.92} = 48{,}000$ lb. above the kneebrace. Similarly, in the outer face of the pier the bracing between the 18-in. I-beam columns was designed for a shear of 25% of 31,500 lb., or 7850 lb. below the kneebrace and 16,000 lb. above the kneebrace. In both cases the kneebrace was designed for a stress whose horizontal component is the sum of the shears above and below.

Personnel and References

The plans for the dome were prepared by the Division of Works of the Exposition Company. H. D. H. Connick is Director of Works and A. H. Markwart Assistant Director of Works. The dome and substructure were designed by the writers under the immediate supervision of H. D. Dewell, Chief Structural Engineer of the Division of Works. After the design was complete, the stresses in the dome were checked by Richard G. Doerfling, Civil Engineer.

Bakewell & Brown, of San Francisco, were the architects of the building. Dyer Bros., of San Francisco, were the contractors for the fabrication and erection of the steel.

The methods discussed are outlined fully in the following original publications, which were consulted:

E. Schmitt, Dome Design. Trans. Am. Soc. C. E. Vol. 52, 1904.
Hazlehurst, Towers and Tanks for Waterworks. Wiley, 1910.
Engineering News, Design of a Gas Holder. Vol. 66, p. 644.
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Greene, Structural Mechanics, 1897.
Eddy, Researches in Graphical Statics. Van Nostrand, 1878.
Hütte, Des Ingenieurs Taschenbuch, Abteilung II. Berlin, 1899.
Müller-Breslau, Die Neueren Methoden der Festigkeitslehre.

Discussion on Prime Movers

The general features of the San Francisco convention of the National Electric Light Association and some of the discussions have been noted in previous issues. The following paragraphs summarize the discussion on prime movers:

The report of the Committee on Prime Movers (I. E. Moultrop, Boston, chairman) took up in succession steam, water and gas power. Comment was made on widespread cracking of boiler plates in districts where carbonate of soda was found in the boiler-feed water. The water-power section consisted largely of a series of questions and answers on the advantages of open and closed hydraulic-governor systems. These showed great diversity of opinion.

The discussion of the report was voluminous. Part of it turned on the benefit of improving boiler-room practice by a greater use of measuring instruments. In discussing hydraulic power the salt-solution test for flow was the important topic. An objection was raised to the method on account of the extremely accurate analysis of the solution necessary; but it was argued in rebuttal that, without any undue refinement in the analysis, a 0.94-per cent. check had been obtained between a rectangular-weir and a salt-solution test of a discharge of 36,000 gal. per min. The costs differed very little. Another member claimed to have secured a check of 0.5 per cent. by measuring the electrical conductivity of the solution, using an instrument calibrated to read to salt concentration directly. A still more ingenious scheme consisted in sending the water through a pipe at one section of which was an electrode connected with a measuring instrument to give a continuous record.

In the paper entitled "Analysis of Waterwheel Governor Effort," by E. D. Searing, of Portland, Ore., some interesting work done by the Portland Railway, Light & Power Co. in studying the operation of governors was described. After eliminating some unsuccessful methods of attack, steam-engine indicators were connected to each end of the governor cylinder of the waterwheel unit under observation and continuous records of varying pressure from each side of the governor piston were obtained throughout cycles of operation. By an analysis of the governor effort, friction and unbalance were separated.

New Heavy Electric Railroad Opened in Michigan

A 2400-volt direct-current high-speed heavy interurban line has recently been put into operation by the Michigan Ry., connecting the cities of Grand Rapids, Kalamazoo and Battle Creek. This line was a project of the late W. A. Foote, of the Commonwealth Power, Railway & Light Co., and was made feasible by what was felt to be insufficient local railway development. A new 50-mi. single-track line has just been completed from Kalamazoo to Grand Rapids. A line from Allegan to Battle Creek, crossing the other at Monteith Junction, was purchased some time ago from the Michigan Central R.R.

Interurban Service

Three types of service are being given—local, express and "flyer." Local trains make the Kalamazoo-Grand Rapids run in 1 hr. 45 min., and the Grand Rapids-Battle Creek run in 2 hr. 25 min. Single cars from Kalamazoo and Battle Creek join at Monteith Junction and run as a train to Grand Rapids.

The express run from Grand Rapids to Kalamazoo requires 1 hr. 25 min., and that to Battle Creek 2 hr. 10 min. "Flyers" run only between Grand Rapids and Kalamazoo and take 1 hr. 10 min. The company previously had an electric line between Kalamazoo and Battle Creek and local systems in the terminal cities. All the cars in the new service are of steel and the "flyers"

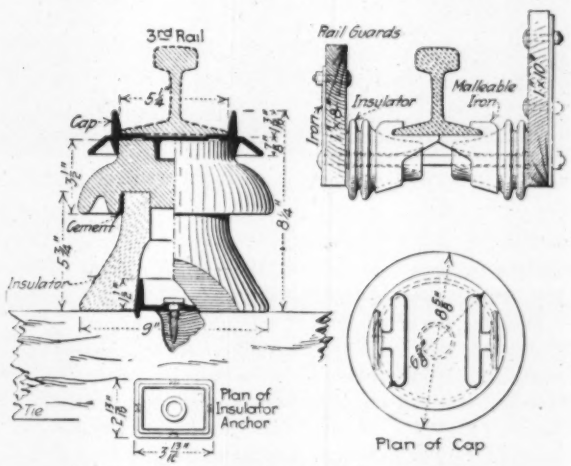
have special luxurious equipment. Bulk freight, package-freight, and express service are maintained, steel motor and trailer cars being used for the last two classes.

At Grand Rapids and Kalamazoo property was acquired for passenger and freight station and yard tracks. At the larger towns along the new road brick and concrete stations were put up; at the smaller places neat wooden buildings were erected, and for the important cross-road stops wooden shelters with concrete platforms.

Roadbed and Track

The new line (single track) was built through rolling country and required a few fairly heavy cuts (up to 85,000 cu.yd.) and fills (up to 60,000 cu.yd.). Except for the Kalamazoo city-street tracks and on the Grand Rapids private right-of-way, the maximum curvature is 3 deg. and the maximum grade 1 per cent. In cuts the roadbed is 24 ft. wide; on fills it is 18 ft. The right-of-way is generally 100 ft. wide and is fenced and equipped with cattle guards. All bridges are of concrete or steel.

The tracks are of 80-lb. rail laid on 6x8-in. by 8-ft. cedar ties, 20 to a 33-ft. rail; each sixth tie is extended



THIRD-RAIL CONSTRUCTION, MICHIGAN RY.

4 ft. to carry the third-rail supports where that equipment is used. Passing sidings are about 1000 ft. long and of main-line construction. Grade crossings over other railways have interlocked home and distant signals on both roads. Telephone jack boxes are installed at passing tracks and a telephone dispatching line is provided into which a conductor may connect.

Electric System

This road has the first 2400-volt over-running third-rail line in this country. Catenary overhead construction is resorted to where the road leaves its fenced-in right-of-way for open stretches in populated districts.

The third rail is a standard 80-lb. A. S. C. E. section rolled of special low-carbon steel (limit of carbon, 0.14; of manganese, 0.40; of sulphur, 0.08, and of phosphorous, 0.11). Slotted malleable-iron fish plates are used, allowing expansion, the joints being 1½ to 2 times as wide open as in the running rails. Copper ribbon bonds bridge the joints. The rail is carried by a malleable-casting cap sitting on a two-section 5000-volt porcelain insulator as shown. The insulator fits over a square malleable anchor held down to the tie by a lag screw. On sidings a 52-lb. rail is used, but the clearances are maintained by special insulator caps. The rail is guarded by two wood side boards supported from the rail by bolts, malleable castings and porcelain bushings. The inner board is made as high as possible without interfering with the collector shoes. The outer board is high enough so that any straight tools dropped fall across the boards and fail to touch the rail. At the highway crossings and in changing from third rail to catenary line, the break in running conductor is bridged by jumper cables.

On account of the self-induction in the steel third rails, lightning-arrester aluminum cells are connected from the line to the ground. The voltage rises on the rail as current is broken, and when it reaches a definite undesirable limit the cells pass current and relieve the stress. When the surge voltage drops, the electrolytic film in the arrester prevents further flow.

Overhead Lines

Where the catenary is used over single-track stretches, a single hard-drawn copper cable (500,000 circ.mil.) is strung with 4-ft. sags between arms on latticed steel poles 165 ft. apart. The poles will carry 1600 lb. vertical load 10 ft. out from the pole and 2000 lb. top pull at right angles to the line. Rigid hangers spaced 15 ft. apart along the messenger support a No. 0000 trolley wire parallel to the track.

On parallel sidings wooden poles are erected and cross-catenaries run to the steel poles and support the minor lines. On the few two-track lengths light steel anchor bridges, designed for a 1200-lb. vertical load, a 12,000-lb. longtitudinal four-point pull, and a 4000-lb. transverse pull, are put 300 ft. apart to carry the messenger cables. On curves the hangers are allowed to take inclined positions, much as in the New Haven type of construction. On sharp curves pulloffs are also used.

A 500,000-circ.mil. copper feeder is carried along with the overhead lines to give the same capacity as the third-rail line. On the sections over single track the feeder is carried by a short arm inside the poles; on double-track sections it is suspended alongside the messenger cable with the same sag.

The right-of-way and highway crossings are posted, to warn trespassers of the danger of high voltage, with blue-and-white enameled signs. The usual engineman's warning signs are also of enameled metal, and on galvanized angle posts for the most part. Special large illuminated signs mark the changes from third rail to overhead, or vice versa.

At present current is fed to the railway line at Grand Rapids and Kalamazoo. A transmission line is being constructed which will allow feeding at Battle Creek. At the present substations the 2400-volt direct current is secured from two rotary converters in series. At Battle Creek a motor-generator set will be used.

The construction of the Grand Rapids-Kalamazoo line and the electrification of the Allegan-Battle Creek line was in the hands of the Michigan Engineering Co. (a concern affiliated with the Michigan Ry.), under the direction of G. L. Erwin, president, and G. A. Wagner, superintendent. The late W. A. Foote had an active part in the design, up to his death a few months ago.

The important electrical equipment was furnished by the General Electric Co. and the third-rail supports by the Ohio Brass Co.

Brooklyn Sewage-Aeration and Activated-Sludge Experiments

By E. J. Fort*

SYNOPSIS—Since the Brooklyn Sewage-Experiment Station was put in operation on Jan. 1, 1914, careful studies have been made of sewage aëration by (1) compressed air passed upward from a pipe grid and shallow gravel bed through sewage passing downward in a deep tank containing nine horizontal disk deflectors; (2) an aërated contact bed or tank; (3) the activated-sludge process, carried on in the same tank as (1), after slight changes in piping. The experiments are described and the results summarized.

The Brooklyn Sewage-Experiment Station had as one of the principal objects of its design a series of experiments on the treatment of sewage and sludge by forced aëration with compressed air. Black and Phelps, experimenting with the sewage at the same location in 1910-11, had found it possible to reduce the oxygen demand of the sewage one-half or two-thirds under various conditions, by

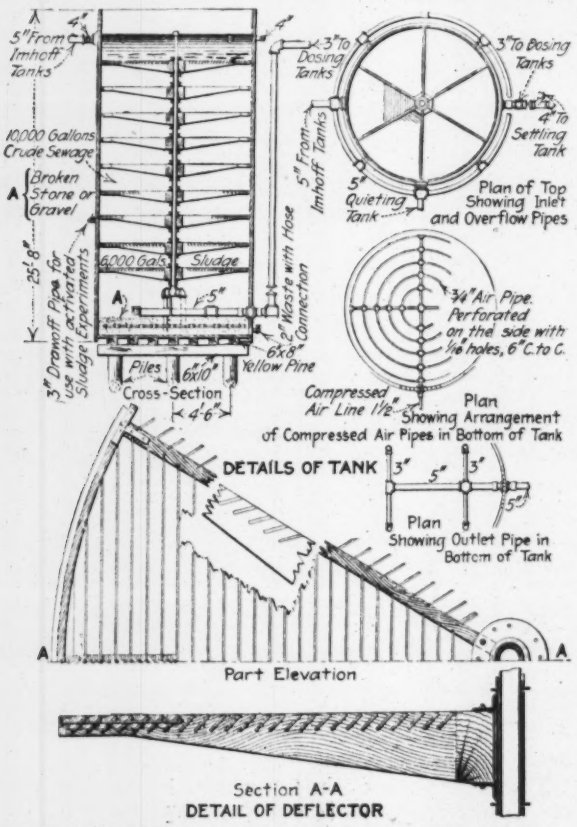

5" From Imhoff Tanks
10,000 Gallons Crude Sewage
Broken Stone or Gravel
6,000 Gals. Sludge
25'-8"
3" Drawoff Pipe for use with activated Sludge Experiments
Piles
6x10"
Cross-Section
4'-6"
3" To Dosing Tanks
5" From Imhoff Tanks
2" Waste with Hose Connection
6x8" Yellow Pine
3" To Dosing Tanks
4" To Settling Tank
5" Quieting Tank
Plan of Top Showing Inlet and Overflow Pipes
¾" Air Pipe Perforated on the side with 1/16" holes, 6" C. to C.
Compressed Air Line 1½"
Plan Showing Arrangement of Compressed Air Pipes in Bottom of Tank
DETAILS OF TANK
Plan Showing Outlet Pipe in Bottom of Tank
Part Elevation
Section A-A
DETAIL OF DEFLECTOR

FIG. 1. DETAILS OF BROOKLYN TANK AERATOR
Used first for plain sewage aëration and then for aëration with activated sludge

applying about two volumes of air per volume of sewage in a retention period of three hours; although the apparatus used was crude and the conditions rather unfavorable for producing the best results. The appropriation to meet the expense of the experiment was very small. While highly gratifying, the results observed were considered by these investigators as demanding a more complete experimental study, which they strongly recommended; and the design and installation of the experimental plant in 1913 gave the opportunity for providing for this purpose equipment and apparatus of special design, with units on a working scale.

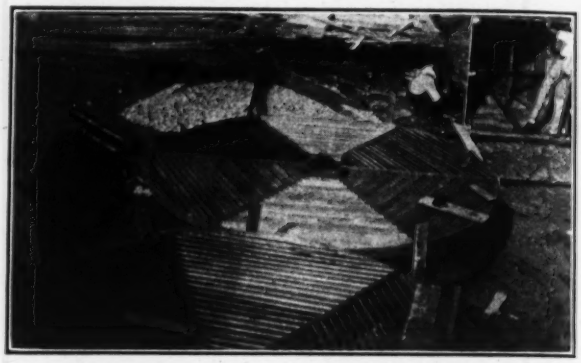


FIG. 2. DEFLECTOR DISKS OF BROOKLYN TANK AERATOR

The plant was completed and put in service Jan. 1, 1914, since which time sewage-aëration experiments have been carried on continuously, except in extremely cold weather. During the experiments, until the beginning of the present year, the principle of activated sludge was not appreciated, although the phenomena were at times noted. Indeed, it is not improbable that the results observed by Black and Phelps were due in some measure to the presence of activated sludge on the grids and deflectors and in the tanks used in their experiments.

The Brooklyn Tank Aerator

The apparatus used in conducting activated-sludge experiments consists of a tank provided for sewage-aëration experiments and listed in the station equipment as the "tank aërator." Fig. 1 shows its size and design. It has an effective capacity of 16,000 gal., sewage being introduced at the top and withdrawn from the bottom; although by a slight change in piping this method of operation can be reversed. The sewage is supplied by gravity from the crude-sewage supply or quieting tank that serves the entire experimental work of the station.[1] It is pumped from the sewer by a reciprocating pump and in passing is retained less than five minutes in the supply tank. Compressed air is furnished by a duplex air compressor of ample size, installed as part of the experimental plant.

The tank aërator had been in continuous use during 1914, except in extremely cold weather, on sewage-aëration experiments, giving some very interesting results, but

*Chief Engineer, Bureau of Sewers, Borough of Brooklyn, New York City. In the preparation of this paper the author wishes to acknowledge the collaboration of George T. Hammond, Engineer of Design, and William T. Carpenter, Chemist of the Sewage-Experiment Station.

[1]The Brooklyn Sewage-Experiment Station was described at length in "Engineering News" of Oct. 22, 1914.

using much more air for the work performed than would be economical in competition with the sprinkling filter for the same degree of purification. In order to adapt it to the purpose of conducting activated-sludge experiments it was only necessary to provide an additional outlet pipe about one-third of the distance up from the bottom of the tank.

The tank is 12 ft. in diameter and 25 ft. 8 in. in height. A grid for supplying compressed air is placed on the bottom of the tank, and 7½ in. of broken stone which passes a 2-in. ring and is retained by a 1-in. ring is provided to support it. The same depth of the same broken stone is placed over it, so that the air supplied to the contents of the tank passes upward at first through the voids in the broken stone. The outlet of the tank, as originally installed for sewage-aëration experiments, is about 1 ft. above the surface of the broken stone, so that in all such experiments about one-twentieth to one-twelfth of the contents of the tank was retained at each emptying. A still lower outlet is provided for draining the lowest level of the tank. The new outlet for the activated-sludge experiments is so placed that 6000 gal. is retained in the tank at each drawing.

The compressed-air supply grid consists of two 1½-in. pipes placed at right angles, forming a cross connected in the center. The arms of the cross are connected with quarter-circles of ¾-in. pipe forming concentric rings, of which there are five. Each ring is perforated at 6-in. intervals with $\frac{1}{16}$-in. holes. The air enters through the 1½-in. pipes and is distributed thereby to the rings and is discharged into the broken stone surrounding the grid. This is intended to prevent any upward streaming effect.

The deflector disk (Figs. 1 and 2) form one of the features of this tank. These disks are nine in number and

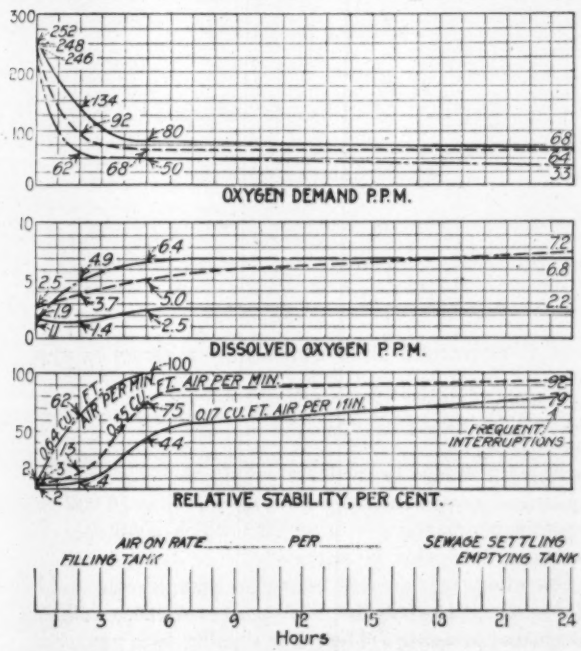


FIG. 3. RESULTS WITH BARREL AERATOR, MAY 6 TO JULY 6, 1915

divide the tank into a number of story-like compartments. They are designed to deflect the downward flow of sewage, so as to prevent streaming downward, and to equalize and give a sinuous motion to the upward flow of air, thus securing a more even distribution of each. The slats are set in grooves in the arms at an angle of 45° with the horizontal. The slats in each alternate disk are set alternately sloping away from and toward the center.

The first experiments with plain aëration of sewage in the aërating tank were not very encouraging, an air supply

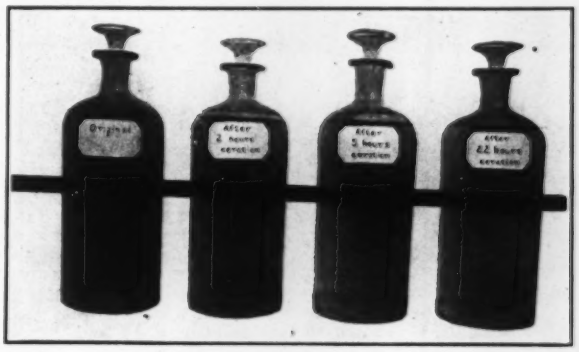


FIG. 4. PROGRESSIVE SEWAGE CLARIFICATION, BROOKLYN BARREL AERATOR

After 0, 2, 5, and 22 hr. aëration. The black band was at the rear of the bottles

of 0.75 volume per volume of sewage with 2 hr. retention being insufficient to produce any marked improvement on crude sewage.

Barrel Aërator or Aërated Contact Bed

A visit to the Lawrence Experiment Station in April, 1914, led to the construction of a small unit consisting of an oil barrel filled with stone, with means of introducing air at the bottom, on the theory that the more area available for bacterial residence the better would be the results accomplished. The effective capacity of the barrel was 20 gal. Experiments with this small unit extending from May 6 to July 6, 1914, are summarized in Table 1 and are shown graphically in Fig. 3.

TABLE 1. RESULTS WITH SMALL AERATED CONTACT BED OR BARREL AERATOR

	Hours Aeration			
	0	2	5	24
Air at 0.17 cu.ft. per min:				
Dissolved oxygen	1.9	1.4	2.5	2.2
Relative stability[1]	2	4	44	79
Dissolved oxygen demand	252	134	80	68
Air at 0.35 cu.ft. per min.:				
Dissolved oxygen	2.5	3.7	5.0	7.2
Relative stability	3	13	75	92
Dissolved oxygen demand	248	92	68	64
Air at 0.64 cu.ft. per min.:				
Dissolved oxygen	1.1	4.9	6.4	6.8
Relative stability	2	62	100	100
Dissolved oxygen demand	246	62	50	33

[1] Interruptions in application of air.

The improvement in the character of the sewage with this treatment is extremely marked (Fig. 3), the 24-hr. samples being almost of drinking-water clarity.

Results with Plain Tank Aërator

Operation of the larger aërator tank was continued while this experiment was in progress, at first on the continuous-flow plan with a largely increased period of retention and a greater volume of air, without effecting any marked improvement in the result obtained. The continuous-flow plan was then suspended for a time, and on May 19, 1914, the tank was filled and air was passed through it for long periods, some of the phenomena observed being those of activated sludge, though the desirability of saving the latter was not recognized. This condition having been established in the tank on June 14, 1914, the continuous-flow plan of operation was again employed with more promising results. It is worthy

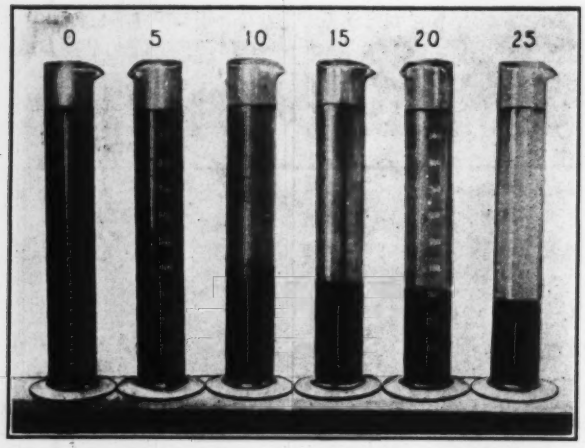


FIG. 5. RAPID SETTLING OF THE EFFLUENT FROM ACTIVATED-SLUDGE TANKS
After 0, 5, 10, 15, 20 and 25 min. settlement in beakers.

of remark, as already mentioned, that about one-twentieth of the contents of the tank, if filled to the water line, is below the discharge orifice; thus at least this proportion of the contents is retained at each drawing. This retained portion would no doubt consist to some extent of activated sludge even when operating on the continuous-flow plan; possibly some of the improvement was due to this fact. The tank was operated with 3.25 volumes of air per volume of sewage with 5 hr. of retention. The quality of the effluent was, however, not satisfactory.

The retention period was increased July 10, 1914, to 24 hr., with the same air flow per minute, bringing up the air to 18 volumes per volume of sewage, with marked effect; the effluent after passing a settling tank with about three hours retention showed a quality comparable with sprinkling-filter effluent. This experiment was carried on until Oct. 1, 1914. In September the effluent was 84% stable. On Oct. 1, 1914, the amount of air was reduced one-half, or to nine volumes per volume of sewage treated, the continuous-flow plan being continued and the retention period maintained at 24 hr. Under these conditions the relative stability of the settled effluent fell to 59 per cent.

Table 2 shows the average of the results of operating the tank aërator from July 10 to Oct. 1, 1914, on the continuous-flow plan.

Until the beginning of 1915 the principle of activated sludge was not appreciated, though, as mentioned in the foregoing, the phenomena were at times observed. In the spring of the present year, however, with the publication of results in which the retention of the aërated sludge played an important rôle, the aërating tank was remodeled by introducing a draw-off pipe in the side of the tank, as previously mentioned, so that after allowing the sludge to settle the supernatant liquor could be drawn off,

TABLE 2. TANK AERATOR RESULTS, JULY 10 TO OCT. 1, 191[illegible]

	Crude Sewage	Tank Aerator	Settl. Tank No.
Settling matter, c.c. per liter	2.3	5.4	0.2
Suspended solids, p.p.m.:			
Total	164	103	40
Volatile	130	79	30
Oxygen Consumed			
Total	58	39	26
Dissolved	37	27	22
Dissolved oxygen	1.4	3.1	1.1
Dissolved-oxygen demand	191	55	31
Relative stability, per cent.:			
Undiluted	2	43	84
Diluted 1-10 with distilled water	35	94	100

Period of retention in tank aerator, 24 hr.
Amount of crude sewage treated in 24 hr., 16,000 gal.
Air applied, 2.3 cu.ft. per gal.
Settlement period of effluent in settling tank No. 1, 3 hr. 12 min.

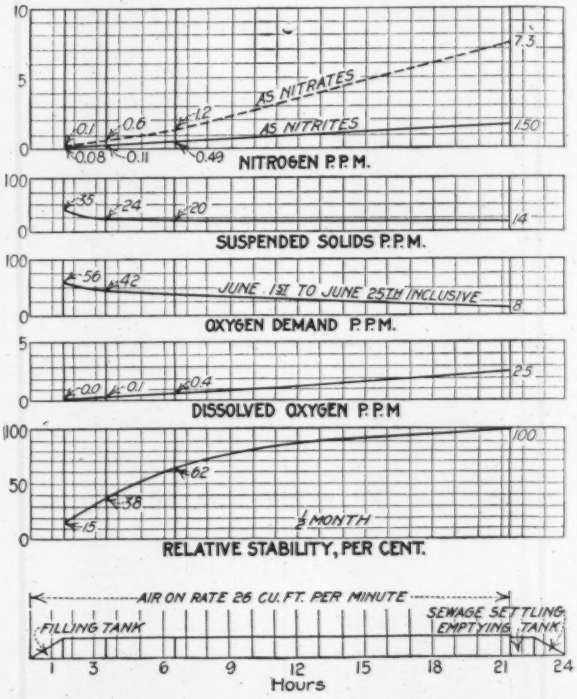


FIG. 7. BROOKLYN ACTIVATED-SLUDGE RESULTS FOR JUNE, 1915

Volume of air per volume of crude sewage, 117, with tank filled; or 26 cu.ft. of air per min. for 10,000 gal. of sewage treated and 6000 gal. of sludge left in tank.

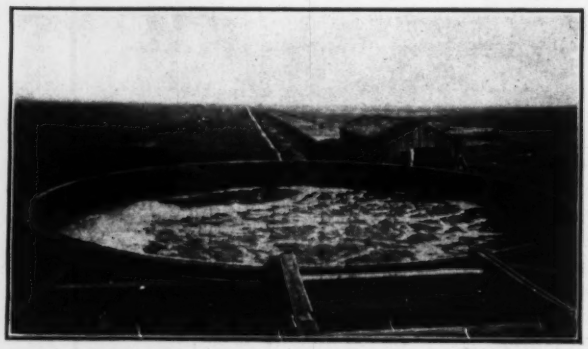


FIG. 6. TOP OF BROOKLYN TANK AERATOR DURING ACTIVATED-SLUDGE EXPERIMENTS

Air is escaping from surface with even distribution and lack of evident boiling

leaving the sludge up to 6000 gal. in the bottom. The amount of sewage treated per charge was thus 10,000 gal. Bottle experiments were also tried which confirmed the experiments of other investigators.

Operations on the large tank were begun in the middle of March, 1915, but the slow process of accumulating sludge and bringing the liquid to stability took practically until June 1. By May the results with 24-hr. aëration showed up excellently.

On June 1 operation was commenced on the following schedule: At 8 a.m. air was shut off and sewage was allowed to settle for one hour. At 9 a.m. drawing off of supernatant was commenced, discharging 10,000 gal. At 10:30 a.m. drawing completed, and air again turned

on. The tank was then refilled with crude sewage. At 12 noon refilling was completed. The sampling was done as follows:

When the tank was filled, a sample of liquid and sludge was collected in an Imhoff cone and the sludge allowed to settle, the dissolved oxygen being determined in the supernatant liquor. Another sample, taken at the same time, was filtered through paper for use in determining relative stability and oxygen demand. A portion of the supernatant was preserved with chloroform for determination of suspended solids, nitrites and nitrates in a weekly composite. Sampling was repeated after 2, 5 and 20 hr., the latter just before emptying the tank.

Fig. 5 illustrates how rapidly the suspended matter goes down when sewage aërated with activated sludge is allowed to settle. Fig. 6 shows the surface of the Brooklyn aërator tank when operated on the activated-sludge plan.

The experiments up to June 1, 1915, confirmed the results of other investigators, with the exception that nitrification appeared not to have proceeded as far as reported previously.

The results for the month of June, 1915, are shown by the diagram in Fig. 7, and by Table 3.

TABLE 3. RESULTS WITH ACTIVATED SLUDGE, JUNE, 1915

	Hours Aeration After Refilling Tank			
	0	2	5	20
Dissolved oxygen, p.p.m.	0	0.1	0.4	2.5
Relative stability,* per cent.	15	38	62	100
Oxygen demand (biologic) † p.p.m.	56	42		8
Suspended solids, p.p.m.	35	24	20	14
Nitrogen as nitrites, p.p.m.	0.08	0.11	0.49	1.5
Nitrogen as nitrates, p.p.m.	0.1	0.6	1.2	7.3

* June 1 to 15, inclusive. † June 1 to 25, inclusive.

The sewage used to fill the tank had the following average composition: Dissolved oxygen (p.p.m.), 1.0; relative stability, 3%; oxygen demand (biologic), p.p.m., 200; nitrogen as nitrites and nitrates, trace.

Commission Revises Water Rates in Leavenworth

An apparently anomalous situation was recently created in Leavenworth, Kan., when the Public Utilities Commission raised the rates for much of the service of the Leavenworth City & Fort Leavenworth Water Co. in response to a complaint of the city that the "rates charged a majority of the inhabitants of the city, and individuals residing outside, are excessive, oppressive, unreasonable and greatly in excess of the fair and reasonable value of the service rendered."

Old Service and Rates

The anomaly disappears when it is understood that the company supplied large quantities of water (1,627,000 gal. per day) to the Federal military reservation, penitentiary and soldiers' home at from 6 to 10c. per 1000 gal., whereas only 20 to 25 per cent. of the city population was served with about 1,450,000 gal. per day (not including some 2,050,000 gal. of free water for schools, public buildings, parks, flush tanks, gutter washing, etc.) at rates of from 50c. to 20c. per 1000 gal.

There was also an apparently haphazard flat-rate schedule covering some forty manufacturing and domestic items. The customers whose supply was metered were required, prior to this proceeding, to purchase a high-priced meter ($15, $17.50 and $21 for ⅝-in. size) on which the company realized from $5 to $7 profit.

Investigation showed that the water furnished the Federal government had to be pumped twice and was specially filtered. Removal of 97% of the bacteria and a minimum count of 100 per c.c. were guaranteed. An investment of $16,250 had been made to carry government sewage below the water-works intake.

The total pumpage was shown to be 1,875,000,000 gal. in 1913, of which 40 per cent. was alleged to be free water furnished to the city.

The total operating expenses and interest was $92,720, making the cost of water sold over 8c. per 1000 gal. About $50,000 more revenue was claimed necessary to put the property on a paying basis; this would make the cost over 12c. per 1000 gal.

Physical and Intangible Values

The present physical value was placed at $497,108, as determined by the Commission's engineer, T. J. Strickler. The company's engineers, Burns & McDonnell, of Kansas City, appraised the property at $834,374. For "going value" $37,283 (7½ per cent.) was allowed, making the rate-basis value $534,391. In making this last allowance the Commission raises this interesting query:

> Here we have a water plant in operation for nearly a third of a century, enjoying a monopoly in its territory, unburdened with the expenses incident to ordinary competition, serving now only one-fifth to one-fourth of the water users of the city of Leavenworth, with prices to domestic consumers, at all times, so high as to be almost prohibitive, and with an average net profit of only slightly over $3000 per year upon a claimed valuation of over $800,000, or an ascertained valuation of about half a million. What, in such a case, should be added to the ascertained present physical value for intangible or going value?

The Commission answers its own query by reviewing the practice of other bodies and concludes that in this case it should be the cost of building up the business where patrons cannot obtain service elsewhere and where they have been willing to lay private lines along the streets.

Rate of Return

A reasonable return was found to be 6 per cent. It is remarked that if the water business of the city had been properly developed, a higher rate might have been allowed, and that if the Commission's orders are carried out in good faith the business will still develop and automatically increase the return.

The new domestic rates start with a minimum charge of 50c. per month and run approximately straight from 28c. per 1000 gal. for the first 10,000 gal. to 6c. for all in excess of 20,480,000 gal.

The flat rates were unchanged on account of the possible hardship which might come to the smallest customers. The city's free services were ordered metered and charged at regular rates—except for water from hydrants used in flushing gutters, which was held covered by hydrant rental. The latter figure ($40 per hydrant per year) was left untouched with the note that if the 150 additional hydrants that were needed were installed, then this rate on 300 would prove compensatory.

The old requirement of guaranteed income of $25 per block as a pre-requisite for an extension of mains was changed to 10% of the cost of the extension (besides hydrant rental). The idea of a guaranteed return on a proportion of plant and other investments was rejected.

It was held reasonable that the customers should purchase their meters from the company, provided there was "proper regard for the patron's interests."

Field and Office

Model Record of Distribution of Construction Plant

It is important for contractors doing work at numerous and widely scattered points, and having a large amount of equipment distributed at these points, that they should have some system of keeping track of the location and movements of such equipment. A comprehensive and picturesque system which has been adopted by the Great Lakes Dredge & Dock Co., of Chicago, consists of the use of models placed in glass-covered display cases and arranged to show the character, location and condition of the equipment. This is operated in conjunction with the plant inventory, which takes care of everything, from small tools to large pieces of equipment. The system originated with William A. Lydon, president of the company, and *Engineering News* is indebted to the company for the information.

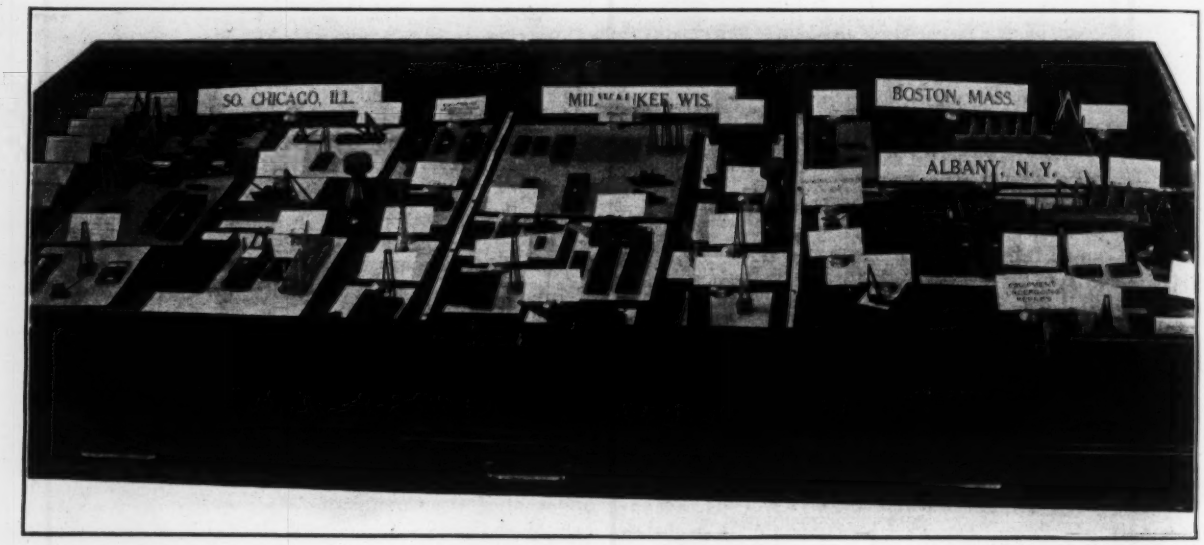


FIG. 1. CASE OF MODELS FOR RECORD OF CONSTRUCTION PLANT; GREAT LAKES DREDGE & DOCK CO., CHICAGO

In the main office at Chicago there are four mahogany display cases, 7x4 ft., placed on tables and covered with ¼-in. plate-glass tops, which are made in sections to allow for ready removal. One of the cases is shown in Fig. 1. The inside of the case is painted black, and the bottom is divided into sections by ½-in. strips of white cardboard. Each section represents one port or harbor, as shown, and its size depends on the amount of equipment at that port and the corresponding number of models required to be placed in the section. The system is for movable plant only and does not cover such stationary units as machine shops, concrete-pile plants, coal docks, quarries, etc., the location of which is permanent.

The models are grouped upon sheets of cardboard of three colors: Those placed on a white card indicate plant that is working; those on a green card indicate plant that is idle, and those on a salmon card indicate plant under repair. On the green and salmon cards are placed small stands or easels, each bearing a card lettered "Idle" or "Equipment undergoing repair." Each piece of equipment under repair has an "A.F.E." card—that is, showing "authority for expenditure."

A red card, or "danger signal," about 3x3 in., is used to represent "Work awarded but not yet started." As soon as work is started the red card is replaced by a white card. Each particular contract in each port is represented by a 2x3-in. white card, placed on an easel and bearing the contract number and a typewritten description of the work. A typical example of one section of a case, showing the record of one port, is given in Fig. 2.

The models are of wood (mostly cherry) on a scale of 20 ft. to the inch. Those representing steel equipment are painted gray, while those representing wood are of natural color, varnished. Each model shows the main characteristics of the machine it represents. Thus the drill boat under "Boston," in Fig. 1, is varnished and has the drill frames painted gray, indicating that it is a timber hull with five steel drill frames. Models of dipper dredges have a projecting boom (appropriately colored), as shown under "Milwaukee," while those of hydraulic dredges (see "Albany," in Fig. 1) have a cutter frame and A-frame. The models of pile-drivers show at a glance whether they are of the floating or land type, etc. The two pedestals in the center line of the case are simply supports for the glass tops.

On each model information is given as to the name or number, capacity and condition of the machine represented. Thus, a dipper dredge will show the size of clay and rock dippers, the maximum dredging depth and the condition. Dump scows are divided into the proper number of pockets or hoppers, and show the cubic capacity, number or name of scow, and its condition.

The ports and harbors represented in this model system include a large number of those on the Great Lakes, the Atlantic Coast and the Hudson River. The models represent about 600 pieces of movable plant. The cases are "posted up" every morning by inventory clerks, the models being shifted according to the daily reports of various local superintendents.

Each of the company's ten branch offices in other cities has a similar system, consisting of one case containing models of the plant in that particular district. The models are shifted in accordance with the previously mentioned scheme. Removal of the plant or equipment from one division to another—that is, transfer of supervision of work from one branch office to another—is taken care of by shipping the model of the particular piece or pieces from one office to the other, so that at all times each branch office has a model of each piece of the plant or equipment at work in its district or under its supervision. The general scheme of subdividing under jobs, equipment idle, under repair, etc., is carried out as outlined in the foregoing for the main office at Chicago.

A specially interesting feature of this model scheme is its use in figuring equipment for prospective work. The models being scaled 20 ft. to 1 in., plans are made to the same scale, and plant or equipment is placed on this chart or map so as to indicate proper clearance in channels, bridge draws, etc. Also, dredging units can be built up of models for any proposed job, using the requisite number and capacities of scows to suit local conditions, depth of water, length of tow, etc. This scheme has worked out to great advantage and visualizes the proposed work to the outside men or superintendents better than drawings or written information could possibly do.

That there is an educational side to the system is illustrated by the rapid manner in which it enables new employees, such as superintendents, engineers and others, to become familiar with the equipment and with the situation on the different contracts. A view of the character and number of the different pieces of plant and of the available equipment is obtained by glancing over the layout of the models and creates more of an impression on the mind than any written description. Further, it gives to stockholders and prospective customers an excellent idea of the extensive amount of equipment and the wide extent of the company's field of operations.

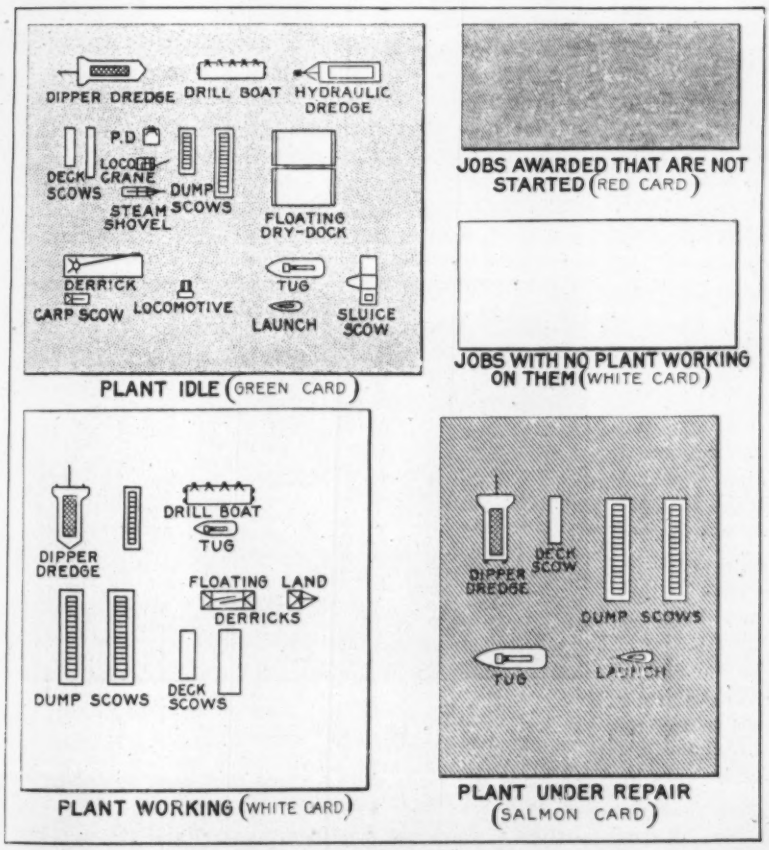


FIG. 2. TYPICAL PLAN OF CONTRACTOR'S MODEL RECORD OF CONSTRUCTION PLANT

Equipment represented by models on the white card is in service. That shown on the green card is idle. That shown on the salmon card is under repair.

Method of Adjusting Sewage Sprinklers

By A. T. Nabstedt*

When placing a new sprinkling filter bed in operation the question of distribution naturally assumes a rôle of vital importance. The following experience, which the writer recently had with a municipal plant of several acres, may be of interest to others. At any rate, this method of approaching the problem is offered for the purpose of soliciting criticism, so that, if possible, varied experiences may be brought together and a more perfect method of correction be applied.

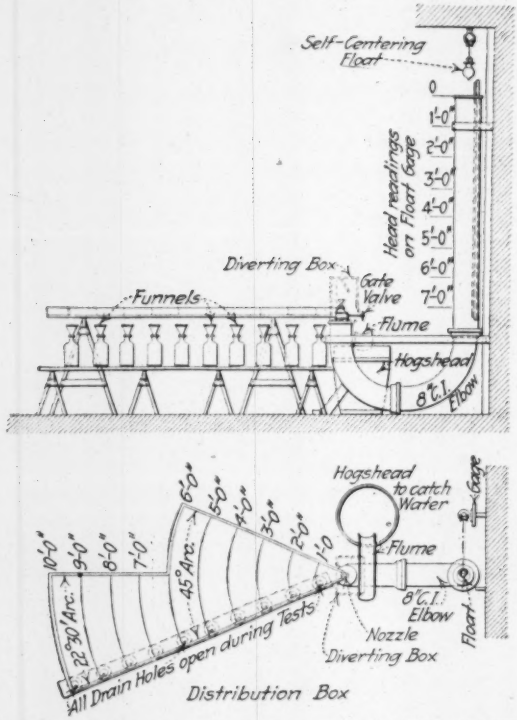
It was quite apparent from the beginning of the operation of the plant in question, that the distribution over the allotted areas would be quite unsatisfactory. Yet a great amount of care was used in the design of the plant, and the best available data on the sprinkling capacities of the various nozzles were at hand.

No mention will be made here of the methods used to build up gradually the available head and of the corrections applied to the various losses of head, but the description will be limited to the application to the spraying device of the best head available and the method used for improving the distribution under the conditions thus presented.

Under what were believed to be the best available conditions as regards the operating elevations of the siphon and the conservation of the hydrostatic head to the nozzle, a windshield was carefully erected around typical nozzles in actual service and a series of distribution tests was made. In these tests the same radial distribution box was used, which will later be described in connection with the interior testing apparatus, and the mean of a series of five successive tests was accepted.

*Associated with A. L. Webster, Consulting Sanitary Engineer, 80 Maiden Lane, New York City.

Having thus quite carefully found the discharge and the distribution of the spraying device under the above-mentioned conditions, it was the purpose to duplicate the same on a testing apparatus, so that the conditions of the field could be applied to one nozzle and the changes in design, capacity, etc., of the one nozzle be applied back again to the field of 1500 or 2000 nozzles.



APPARATUS FOR TESTING SEWAGE-SPRAYING NOZZLES

It was thought that all conceivable changes could thus be applied to a small unit as representing the entire field, and when the best changes had been accomplished these could be applied to the field at large. The advantage of this indirect experimental procedure over applying the same changes to a large number of nozzles on a field is obvious, whereas the changing of one nozzle among a thousand by testing directly on the filter beds would, no doubt, be open to criticism.

To duplicate the exact conditions of the field on the interior testing apparatus, the actual head on the spraying device was very slowly decreased from the theoretical head supplied by the siphon, and at the same time the capacity of the standpipe was decreased by placing timbers of various sections in it. Finally, a distribution was produced which was comparable with that shown by the tests on the field. It was under these conditions that the changes were applied to the capacity and design of the spraying device.

The accompanying plan and elevation of the testing apparatus make a description of its operation scarcely necessary, and it will, therefore, be but very briefly reviewed.

With the float-gage set at the desired head, the 2-in. gate valve was opened and the rate of discharge of the nozzle was determined by the rate at which the indicator rose foot by foot on the gage board. When the elevation or head was reached which represented the low-water level or venting point of the siphon, the gate valve was closed. The personal factor in opening and closing the valve was partly overcome by carefully considering the time taken for the operation and by imitating the initial and final sprays of the filter beds, as far as that was possible. A quick-opening valve might not quite give this same effect.

The spray, as it fell over the tin-lined distribution box, drained into the respective receptacles very quickly, while the box, which is designated on the accompanying sketch as being set over the nozzle, served the purpose of limiting the spray to an area slightly greater than that of the distribution box and deflecting the remainder of the discharge backward into the hogshead. In this way approximately 90% of the water was saved for the next test.

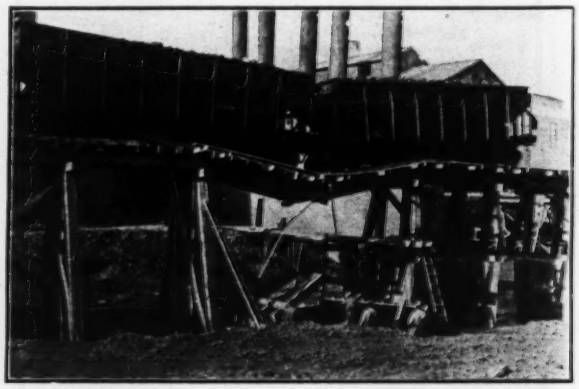
Where a large number of tests are to be carried on such facilities as have been described are appreciated.

The results of these tests not only gave an exact knowledge of what the capacity of each of the various designs was under each foot of head applied, but also its spraying ability with these same heads. Incidentally the results pointed out the exact loss of head in the filter system.

Underpinning a Trestle

Underpinning work on a coal trestle and a little accident which happened during this work are shown by the photograph herewith. The trestle is about seven years old. In spite of the fact that the sills rest on masonary footings, the lower portions of the bents were rotting away where coal had been stored for long periods. In planning repairs it was decided to use concrete piers in place of wooden bents, with a view to protection from coal fires. However, the poor business outlook last summer led to cutting down the cost of the work to a minimum, and for this reason the piers were built up only far enough to displace all bad timber, the remainder of the height to be built at some later period.

The work had to be done without interruption to service. It was carried out as follows: Timbers were



A TRESTLE ACCIDENT; HOPPER-BOTTOM CARS PREVENT COLLAPSE

bolted to the bents above, where they were to be sawed off, and jacks were put under these timbers. The bad portion of the bents was then removed and the concrete was placed and allowed to set several days before removing the temporary support. Old iron rods, pieces of pipe, etc., were placed vertically in the top surface of

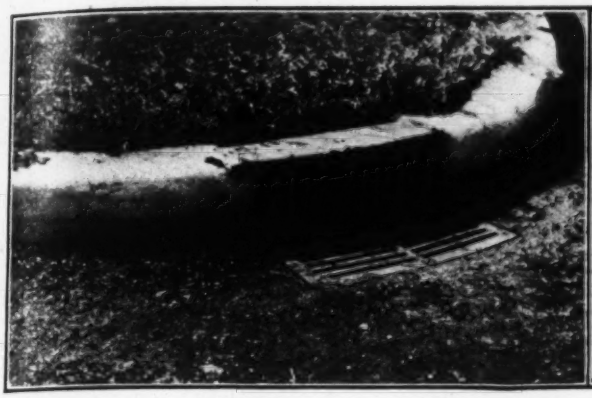


FIG. 1. INLET AT CURB CORNER ON UPHILL SIDE

FIG. 2. OUTLET IN CURB BELOW STREET INTERSECTION

the concrete to give a good bond when the piers are extended up to the stringers later.

The accident shown in the photograph occurred in a peculiar way. A bent had been jacked up and sawed off, when a string of loaded coal cars was pushed over the trestle. The ground supporting the jacks sank slightly under the load, and jacks as well as bent fell to the ground, leaving the stringers hanging in the air, held only by the rails, which did not break. The hopper bottoms of the two coal cars over the break came down to a bearing on the rails, and this saved the day.—A. C. Hewitt, Hagerstown, Md.

Taking Care of Drainage at Street Intersections

By Clair V. Mann*

An arrangement of inlets and conduits for surface drainage at street intersections, somewhat different from that at Inglewood, Calif., described in *Engineering News*, Mar. 5, 1914, is giving satisfaction in Boulder, Colo. This drainage system has been constructed in Grant Place, which has a grade of from 7 to 9 per cent. The grades of cross-streets vary from ½ to 3 per cent.

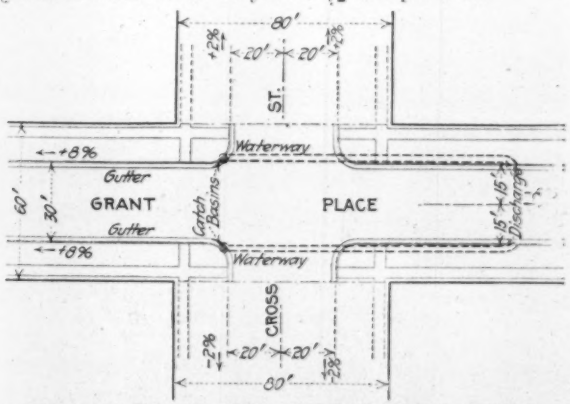


FIG. 3. PLAN OF SURFACE DRAINAGE CONDUITS, BOULDER, COLO.

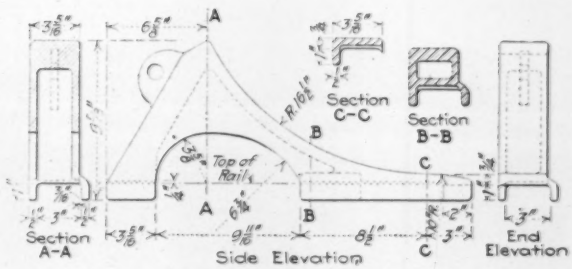
Inlets (Fig. 1) and shallow catchbasins were provided at the curb corners on the uphill side of the cross-streets. Leading from these catchbasins, under the cross-streets and the sidewalk parking area beyond, there are concrete conduits with covers made in removable sections. The grades for these conduits are sufficient to prevent deposit of silt, and such that they meet the grades of the gutters on the downhill side of the cross-street within 50 or 60 ft. beyond the lower line of that street. At this junction point the conduit is brought through the curbing to join the gutter, as shown in Fig. 2. A thin cast-iron cover, for the lower end of the conduit in the parking area, is shown raised up to permit cleaning the outlet. Fig. 3 is a plan of the layout.

*Civil Engineer, Boulder, Colo.

Portable Stop for a Car-Ferry Incline

The Chicago, Burlington & Quincy R.R. is operating a car ferry on the Ohio River for its new line between Metropolis, Ill., and Paducah, Ky. At each end the shore track is connected to the boat by an incline with a cradle that is adjusted to meet varying river level.

To hold the cradle in position on the incline, and also to hold the train on the cradle while its track is being adjusted to the level of that on the boat, portable rail stops are used. The stop, shown in the accompanying drawing, is of cast steel 23¾ in. long and 9⅝ in. high, weighing about 50 lb. It has a grooved base to hold it



PORTABLE STOP FOR CAR-FERRY INCLINE; CHICAGO, BURLINGTON & QUINCY R.R.

in place on the rail, and its bearing surface is corrugated in order to get a good grip on the rail head. The toe is feather-edged to engage the wheel, which rolls up on the stop against a surface having a radius of 16½ in., or about that of the wheel tread.

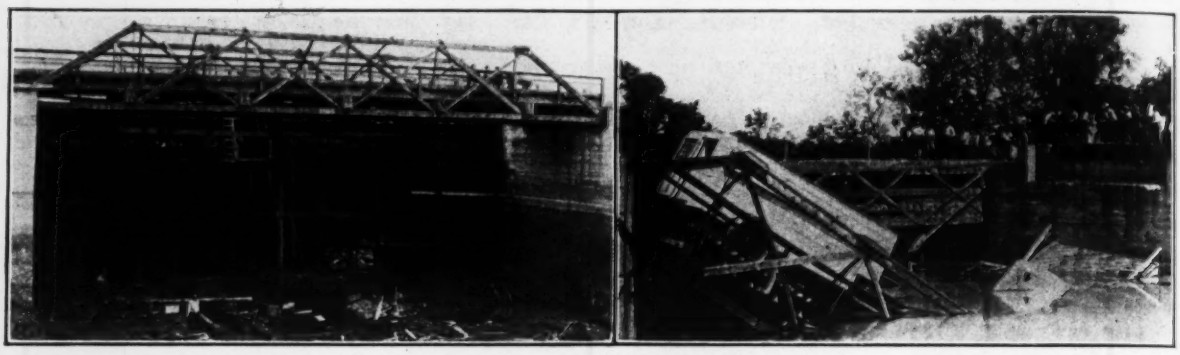
The U. S. Reclamation Service up to Dec. 31, 1914, has built dams and canals requiring a total of earth and rock excavation amounting to over 122,000,000 cu.yd. The total length of canals and ditches constructed by the Reclamation Service is more than 9000 mi.

Collapse of Unstiffened Timber Howe Bridge

An 80-ft. wooden bridge owned by Reclamation District No. 108 and spanning a reclamation canal near Knight's Landing, Yolo County, California, failed May 15, 1915, as noted in *Engineering News*, June 3, p. 109. The bridge was built only last fall. It is said to have been designed to support a load of 30 tons. At the time of the failure it was supporting a small tractor of the caterpillar type, followed by a flat wagon with a small load and also by a cook car. The total load was 11 tons.

The bridge was part of a floodgate separating a drainage canal from the Sacramento River. The trusses were low (pony), and were of the Howe type with total span of 80 ft. Both upper and lower chords were laminated and were in each instance built up of four 2x10-in. pieces. The main diagonals were 8x8 timbers. The floor-beams came at the hanger-rods, so that the trusses were practically unstayed in the top chord.

At the time of the failure the tractor had passed two-thirds of the way across the bridge. Both upper and lower chords failed completely. Eyewitnesses were unable to say whether the upper or lower chord failed first. The two trusses failed almost simultaneously.



FIGS. 1 AND 2. A SHORT-LIVED WOODEN BRIDGE
Left—Bridge practically completed, Dec. 26, 1914. Right—Just after failure, May 15, 1915

Precise-Level Benchmarks, New York City

There has recently been completed a net of precise levels covering the whole of Greater New York. The methods employed on the work were comprehensively described by Frederick W. Koop, Assistant Engineer, Board of Estimate and Apportionment, in *Engineering News*, Sept. 4, 1913. The drawings on p. 223 show the types of benchmarks used, which with the following notes are from Mr. Koop's recent report on the work.

Marks on large permanent structures are considered the most lasting and easy to locate. Such a benchmark is a large T or a + of uniform width with V-shaped indentations at least ¼ in. deep, or a bolt or brass tablet set in the masonry. The difficulty of taking readings on such benchmarks is offset by their permanence and freedom from interference. Readings on such marks are made easier by the use of a hand level, as shown in Fig. 2.

Ground monuments with covers were found to be often inaccessible in winter due to the difficulty of removing the cover; so a plain granite monument with a heavy base of concrete, set with its head about ½ in. above the surface, was found more satisfactory. However, benchmarks

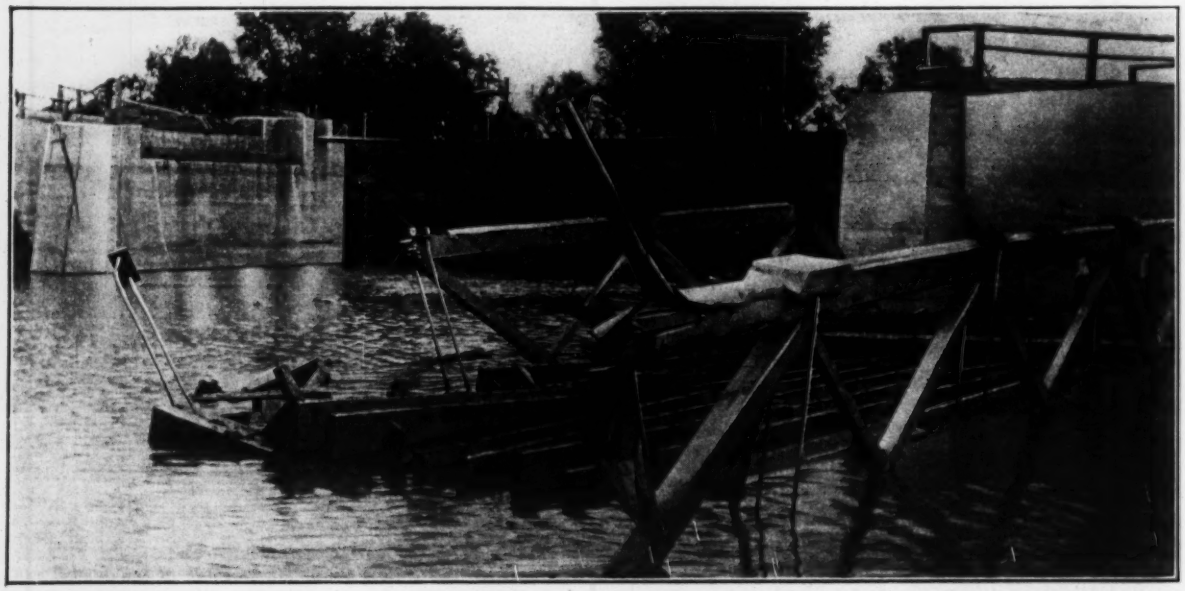


FIG. 3. BROKEN TRUSSES OF KNIGHT'S LANDING BRIDGE AFTER BEING HAULED TO CANAL BANK
Break in near truss is one-half panel from hip; break in far truss is at midspan

projecting any appreciable amount are undesirable on precise-level lines.

With the benchmarks located on buildings, etc., care was taken to place them on the foundations or base courses of massive structures whenever possible, and benches were not established on copings except when provision could be made for tying them into more permanent points in the vicinity. Benchmarks on bridges were either placed as low as possible or were located on the bridge seat. In consequence of the action of successive frosts on the joints of masonry, it was deemed wise, whenever it was necessary to place benches in the immediate vicinity of the waterfront, to select for their location the inshore portions of the structure or wall at points as remote as possible from the water.

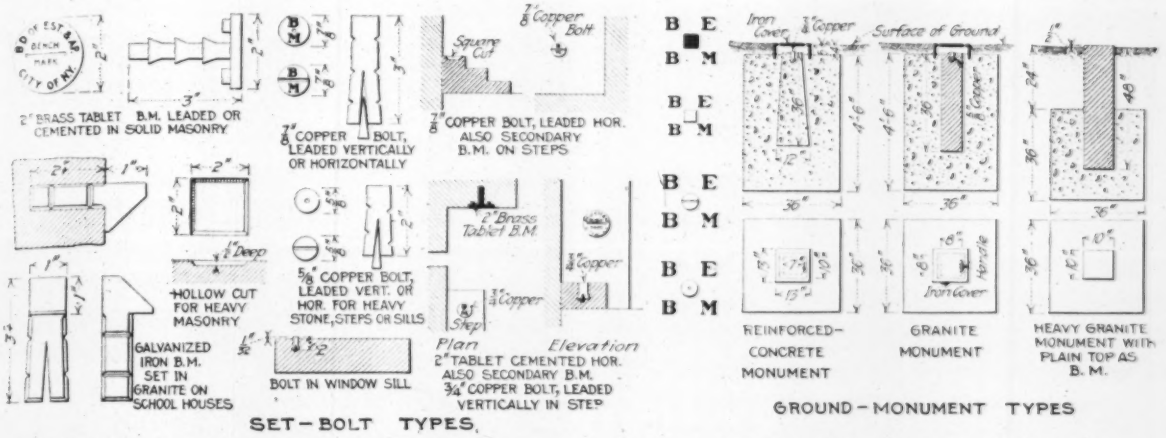

2" BRASS TABLET B.M. LEADED OR CEMENTED IN SOLID MASONRY
7/8" COPPER BOLT, LEADED VERTICALLY OR HORIZONTALLY
7/8" COPPER BOLT, LEADED HOR. ALSO SECONDARY B.M. ON STEPS
HOLLOW CUT FOR HEAVY MASONRY
5/8" COPPER BOLT, LEADED VERT. OR HOR. FOR HEAVY STONE, STEPS OR SILLS
GALVANIZED IRON B.M. SET IN GRANITE ON SCHOOL HOUSES
BOLT IN WINDOW SILL
Plan
Elevation
2" TABLET CEMENTED HOR. ALSO SECONDARY B.M.
3/4" COPPER BOLT, LEADED VERTICALLY IN STEP
SET-BOLT TYPES.
Iron Cover
Surface of Ground
REINFORCED-CONCRETE MONUMENT
GRANITE MONUMENT
HEAVY GRANITE MONUMENT WITH PLAIN TOP AS B.M.
GROUND-MONUMENT TYPES

FIG. 1. STANDARD BENCHMARKS OF THE BOARD OF ESTIMATE AND APPORTIONMENT, NEW YORK CITY

Constructing a 70-Mi. Water Conduit in Alaska

The construction of a conduit comprising 21 mi. of flume, 37 mi. of open ditch, and 12 mi. of pipe was the subject of a paper recently presented by H. H. Hall before the San Francisco Association of Members of the American Society of Civil Engineers.

The conduit leads from the Ogelvie range of mountains southwesterly to the Klondike mines near Dawson City, Alaska. Its purpose is to convey 125 sec.-ft. of water, or about 80,000,000 gal. per day. The conduit was constructed under separate organizations during three seasons of about 120 days each (1907-1909). Ditch construction was used wherever possible—7 ft. wide on bottom, 1½:1 slope, and 3½ ft. deep, with a gradient of 4 to 7 ft. per mi., the greater portion being 6 ft.

The right-of-way was first cleared of trees and the moss scraped off, as fast as it thawed, with teams and slip scrapers. The latter work presented the most troublesome problem from a construction standpoint owing to the uneven thawing of the muck and the consequent miring of the horses. Thirty miles of ditch was dug with six 30-ton steam shovels, the remainder with horse scrapers. As the shovels required a working space at least 16 ft. wide, no side-sloping was done with them. Small patches of frost were removed with 40 per cent. dynamite, but the larger patches were passed over, and after thawing were taken out with scrapers.

The record performance for one shift was 410 ft. of ditch, or about 1200 cu.yd., with an average performance of 14.7 ft., or 51 cu.yd. per working hour. The season was from Jan. 15 to Oct. 15, with work usually slow the first and last weeks. The finishing gang followed the shovels—truing the grade, sloping the banks, clearing the berms, building a path along the top of the soil bank for watchmen, and constructing turnouts and overflows alternately every half mile. Common labor was paid $4 per day and board.

Mr. Hall stated that under operation the ditch had proved satisfactory, leaks being caused usually by ice seams.

Flume construction was adopted where the side slopes exceeded 15 deg. or where the muck was over 2 ft. thick.

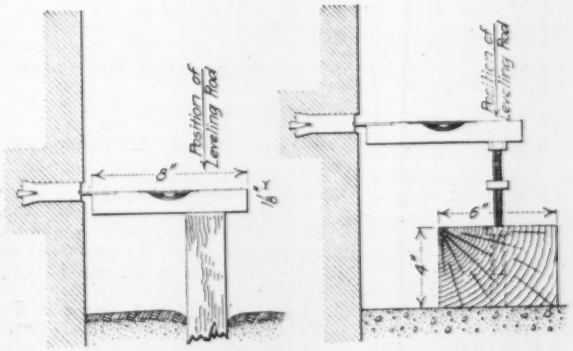


FIG. 2. HAND LEVEL FOR TAKING READINGS ON WALL-PLUG BENCHMARKS

The flume construction was of the board-and-batten type, 6x4 ft. in cross-section, and built of native lumber, made up in 16-ft. sections and set on trestle bents of hewn timber placed 8 ft. on centers. Most of the bents were carried on mudsills, laid below the black muck (about 4 ft. deep), great care being taken not to disturb the moss blanket unnecessarily, and to replace it as quickly as possible over the backfillings in order to preserve the ground in a frozen condition.

It was attempted to found some of the bents on piles driven into earth that had previously been thawed out by means of 16-ft. miner's steam points. In this manner the moss was broken only to admit the pile. Where the

depth of muck would require very deep mudsill pits, the experiment was tried of placing gravel ballast directly on top of the moss and laying the flume stringers on cross-ties.

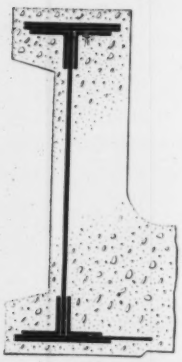
According to Mr. Hall, several years' operation of this flume has shown that the pile foundation is the best. Mudsill bents show continual settlement, caused by water flowing down the posts (either from the flume or from natural drainage) and thawing out the substrata.

Pipe lines were employed for pressure sections. For pressures not exceeding 200 ft. redwood stave pipe was used; for over 200 ft. or less than 1000 ft., standard-riveted; for over 1000 ft., lap-welded steel pipe. One section was constructed of spiral riveted pipe with patent bolted couplings.

In operation it has been found that the wood pipe has given considerable trouble, owing to the fact that between Oct. 1 and June 1 lines are drained and closed up. The sand and gravel carried by the ditch water have worn the bottom staves badly. All intakes are now provided with sand boxes. Accumulations of ice at the discharge ends have caused considerable trouble by increasing the pressure on sections of the pipe. On high-pressure lines the sediment cuts the steel rapidly, and in one instance a ¾-in. rivet was cut away in two weeks.

NOTES

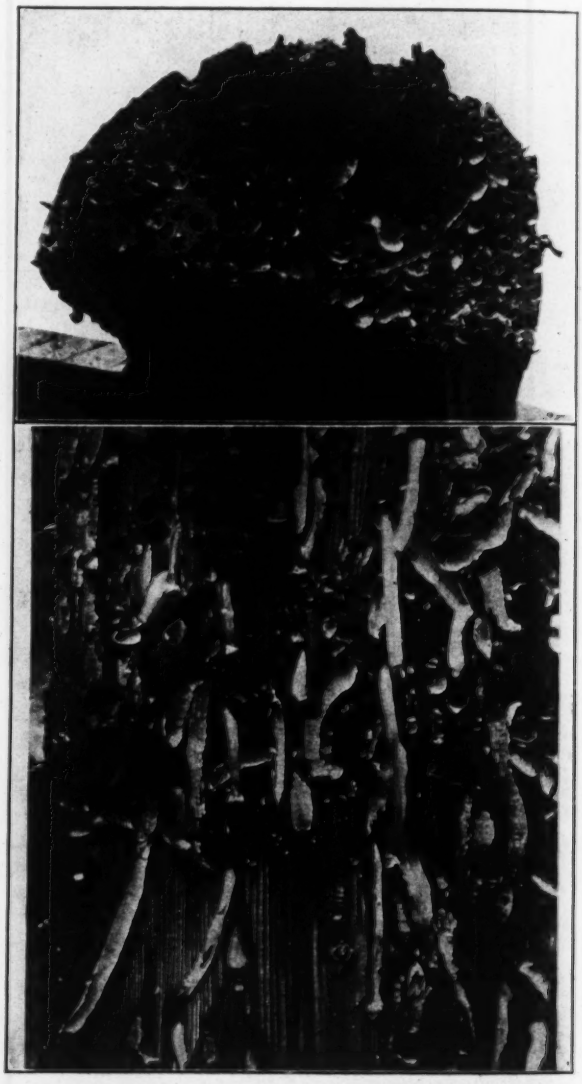
The Slope Angle of Paneling of a concrete-incased steel plate-girder in Chicago has proved troublesome during winter months because snow collecting on the lower ledge melted and dripped down on pedestrians below. The inset here was evidently too deep and the slope too flat, but the designer of the bridge does not know just how much to change both details to make sure that the snow will not lodge in quantity sufficient to cause a nuisance when melting. The angle of friction of snow on concrete evidently enters into the problem, and this angle is probably variable, depending on temperature. At least the incident is of interest to designers of similar structures.



Quick Methods of Excavating—During the recent high water in Kansas, when the Kaw River rose to a gage of 24.6 ft. above standard low water, the drainage board, realizing the immediate necessity for removing all possible obstructions from the river channel, ordered work to be begun at the Union Pacific bridge with the object of removing about 10,000 to 15,000 cu.yd. of earth near the east span. This was accomplished by a gang of eight men drilling and shooting the dirt, using just enough 40 per cent. dynamite to allow it to be broken up thoroughly and loosened. As soon as the shots at one end had been set off, a fire hose, with 1⅛-in. nozzle and about 80-lb. nozzle pressure, which was imparted to it by the use of a single fire engine at the end of about 1000 ft. of hose, was utilized to wash this loosened earth into the swift current of the Kaw. The current of the river at this time was flowing at the rate of approximately 8 to 10 ft. per sec., and the stream from this hose very quickly cut off large pieces of excavation and caused them to be washed downstream. The city of Kansas City, Kan., donated the services of the fire apparatus and firemen to operate it, while the Kaw Valley Drainage Board paid for the shooting. It has not been possible to get a unit cost from this work, but it is needless to say that it was done at a very much lower cost than it could have been done by any other means. Removing spoil banks and earth lying between the levees by means of hydraulic jets during periods of high water has proved to be a very efficient and economical process. The entire amount of the above-described job was removed in about two days time.

A Warning to Chainmen—In an entertaining paper on "Reminiscences" in the 1914 "Proceedings" of the Association of Ontario Land Surveyors, A. L. Russell gives an anecdote which impressed him very early in his career with the responsibility of a chainman: He writes: "I recollect a test made by the late Major Webb, who, as the chain was passing under his instrument one day, closely noted the exact distance to the plumb line. Then saying he wanted to be quite sure of the exact distance from the last tally, he asked the chainmen to repeat the measurement; and when they were out of sight he moved the transit ahead a few links. The head chainman had also noted where the plumb-bob had been on the chain, and gradually pulling the rear chainman ahead to a similar reading, sang out 'Pull right!' and 'All right!' The major remarked that such work was 'too good for anything' and asked them to sign the payroll and clear out."

The Activities of Teredo Navalis in the waters of Union Bay, Vancouver Island, B. C., are well shown by the accompanying two views of a Douglas fir pile recently taken from a wharf there after 19 months' service under water. The upper view shows the head and the lower the exposed center of a split pile, which is characteristic of a number that were replaced.



A FIR PILE AFTER 19 MONTHS' EXPOSURE TO TEREDOS

In the lower view several of the sea worms may be noted with their bulb heads protruding from the little holes they have bored. One of these borers measured 16 in. in length. The views were sent by D. O. Lewis, District Engineer, of Mackenzie, Mann & Co., Ltd., Vancouver, B. C. They were taken by C. E. Sherwood, timber inspector of the company.

Editorials

Mechanical Log Driving

The design of mechanical means for driving logs past hydro-electric and water-storage works is one of the important incidental problems of many such hydraulic developments. A considerable number of mechanical logways are to be found in this country, but comparatively little has been published about them. Therefore, two articles in this issue are of uncommon interest. Details of the provisions for getting logs around the Arrowrock Dam are described in one case; in the other are noted the economic advantages of mechanical over hydraulic handling of the Connecticut River log drives at Turners Falls.

Ordinary sluicing past the dam was out of the question at Arrowrock because of the great height of the structure and the varying reservoir levels. Resort to a mechanical plant was the only possibility. The rugged ground made necessary a novel use of a descending conveyor—the restraining "bull chain" described in the article. This is undoubtedly one of the most rugged conveyors to be found anywhere today.

At Turners Falls to have continued the time-honored practice of shooting the logs through the rapids would have cost the power company the revenue from 1000 hp. of its plant capacity. By carrying the logs through the smooth waters of its power canal, and sluicing them past the power house, the first economy comes—both to the power company and to the lumber concerns. The latter benefit through a great saving in labor, for driving through rapids and loosening the inevitable jams require a considerable force of men. By using a simple snatch-line or a double chain lift only an insignificant amount of water is needed—just enough to wet the sluice—and further economy results.

Shale as a Dam Foundation

The tests to determine the frictional resistance of shale to sliding, described in last week's issue of *Engineering News*, are worth the careful study of every engineer interested in dam construction. A word of caution, however, seems advisable with regard to interpreting these experiments as a guide to practical design.

If their results could be taken at face value, the engineer might conclude that shale was a very safe and satisfactory foundation on which to base a gravity dam. The least coefficient of friction for the sliding of either shale on shale or concrete on shale was well above 0.50. If such a coefficient of friction could be relied upon, the engineer could found his dams on shale rock with entire confidence; but the actual failure of a number of dams founded on shale, notably the great calamity at Austin, Penn., points to the unreliability of the frictional resistance in shale dam foundations.

It is not difficult to explain this unreliability. For example, there are occasionally found in shale rock very thin layers of other materials, such as clay, which may act as a lubricant between adjacent layers and facilitate slipping, if slipping is possible. Again, if there is percolation between horizontal strata of a shale rock the water passing through may gradually erode and dissolve the material until a slight settlement occurs, and this may furnish an initial motion which will reduce the friction coefficient.

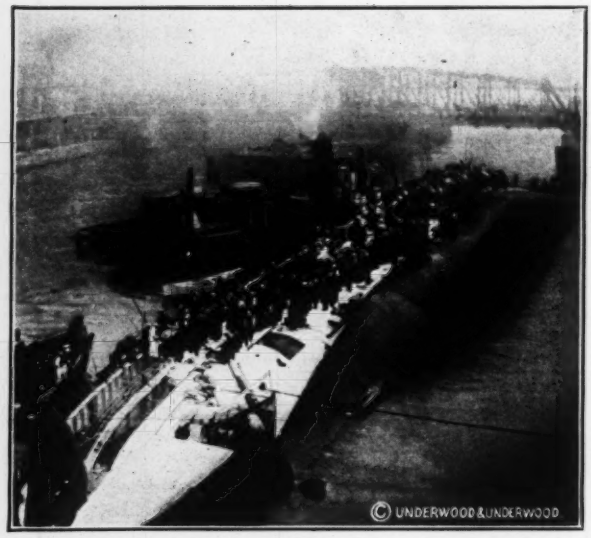
The lesson which we believe most engineers engaged in dam construction have drawn from the numerous failures that have occurred during the past dozen years is that as little reliance as possible should be placed upon the coefficient of friction in the foundation of dams. The bottom of the dam should be keyed to the bedrock by deep foundation trenches filled with good concrete. By this means and by stepping or sloping the foundation surfaces at right angles to the line of pressure it is generally possible so to design a structure that variations in the coefficient of friction of the bedrock surface, even though these variations may cover wide limits, will not seriously affect the safety of the structure.

The Capsizing of the "Eastland"

The terrible disaster in the Chicago River on Saturday, July 24, when the capsizing of an excursion steamer caused the death of nearly 1500 persons, takes rank with the loss of the "Titanic" and the sinking of the "Lusitania" among the most fatal marine disasters ever recorded. The two great transatlantic steamers, however, were sunk in the open ocean, where those who went to rescue survivors had great distances to cover. It would not have been deemed possible that a steamer capsizing in a narrow river, in the heart of a great city, in water so shallow that only part of the hull was submerged and with instant means of rescue at hand could prove as fatal a disaster, in the number of persons carried down to their death, as did the sinking at sea of each of the two great transatlantic liners.

The capsize of the "Eastland" is in fact so unprecedented in marine annals that had such a possibility been suggested by anyone prior to its occurrence it would have been scouted as an impossibility. Here was a great steel vessel built to stand rough weather on the lakes, and consequently with a hull of the regular lake and ocean steamer type of construction. She had a crowd of excursionists on board, but no more than she was licensed to carry and doubtless had frequently carried across the lake. While lying alongside the pier with a stream of passengers coming on board, the boat gradually lists over away from the pier and the list increases until the hawsers holding the boat to the pier are snapped and the vessel rolls over on its side, precipitating the passengers into the river and drowning most of those imprisoned within its hull.

Naturally a unanimous chorous of condemnation has arisen. The public rightly brands such a disaster as inexcusable and demands that the causes be determined



THE CAPSIZED STEAMER "EASTLAND" IN THE CHICAGO RIVER

and that those responsible for this terrible loss of life receive due punishment.

It will be evident at once to every engineer familiar with ship construction that the primary cause of the capsizing of the "Eastland" must be sought for in the condition of the vessel's water ballast tanks. Practically all deep-draft lake and ocean vessels of the present day have water-ballast tanks in the bottom of the hull by which the draft and stability of the vessel are controlled. When a vessel is laden with heavy freight she may have ample stability with the water-ballast tanks empty; when sailing without cargo, however, it is generally necessary to fill the ballast tanks to make the vessel stable and sometimes it is necessary to add extra ballast besides to make the vessel seaworthy in all weather.

In vessels for passenger service, considerations of stability are closely connected with considerations of steadiness and consequent comfort for passengers. If a vessel has too great stability it may be so "stiff" and have such a short period of roll as to be uncomfortable for passenger service. The late Charles H. Cramp, in a paper before the American Institute of Naval Architects and Marine Engineers a few years ago, pointed out that in the endeavor to build steamers with a slow and easy roll, some of the designers of vessels for transatlantic passenger service had gone so far as to build vessels which absolutely depend upon their water ballast for stability and would capsize should their ballast tanks be empty.

In studying the capsize of the "Eastland," a number of the causes which are being assigned by the newspapers for the accident may be at once dismissed. It is said, for example, that the boat was overcrowded. The boat owners and the steamboat-inspection service are assailed for permitting such a number on board. It is also said that the rush of passengers on the upper deck to one side to view some happening caused the careening and capsize of the vessel.

Such circumstances as these are wholly inadequate to explain the occurrence. The vessel had been crowded with passengers many times in its history. On a boat of this type, built without the overhanging guards common on the excursion steamboats used in shallow river and lake waters, it is impossible to throw weight enough to one side by the movement of passengers to have any serious effect on stability, provided the vessel itself is in a properly stable condition.

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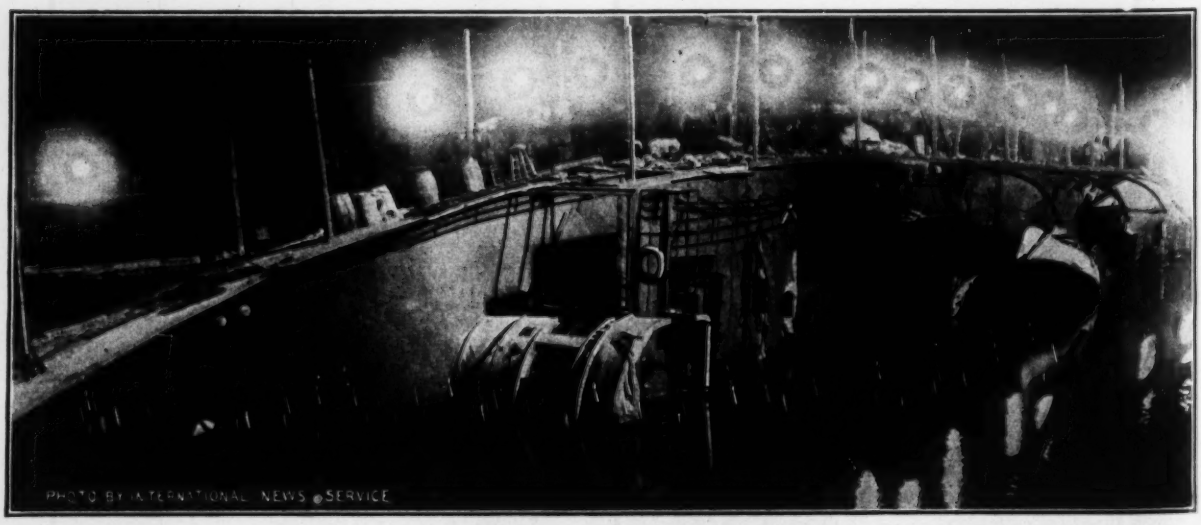

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All these things will doubtless be fully determined by the official investigations. The important thing to the engineering profession is that this unparalleled disaster will call general attention to the little-understood matter of vessel stability and its importance with respect to the public safety. It has been hitherto little realized by the public that the safety of a vessel against capsizing may depend not merely on the correctness of its original design, but also on the intelligence and good judgment of the officers who operate it.

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Serial Bond Issues Desirable

Engineers have been foremost in calling attention to the absurdity of a city or state borrowing money to pay for a public work by a bond issue which will not become due until long after the work for which the money is borrowed has been worn out or has become obsolete. In the New York Constitutional Convention, amendments have already been introduced which limit the maximum term of a bond issue by the life of the improvement for which the bond issue pays.

An innovation in public finance of equal importance, which has also been brought before the Constitutional Convention in proposed amendments, is the substitution, in place of the sinking-fund plan of providing for the payment of bond issues, of the serial bond issue under which an installment of the principal is paid off each year. Of course it makes no difference in theory whether a borrower accumulates year by year in a sinking fund the amount which will be sufficient to pay the debt when it is due or whether he takes the same money each year and pays off a fractional part of the debt; that is, it makes no difference if the investments for the sinking fund and the bond issue for which it is accumulated bear the same rate of interest. Experience has shown, however, that occasionally sinking funds accumulated for the payment of a bonded debt are not kept intact. Losses may occur through unwise investment; or some sudden emergency or deadlock in legislative bodies may prevent an annual payment to a sinking fund from being made.

The plan of making a bond issue payable serially—or, in other words, paying the debt on the annual installment plan—is therefore the safer plan for the borrower, and is safer for the taxpayers as well. There is no possible question in case of a serial bond issue as to the amount of money which must be raised each year for debt-paying purposes; and there is much less likelihood that when a large bond issue matures, the taxpayers of that day, who may be the children or grandchildren of those who originally created the debt, will find that the funds which should have been accumulated to provide for the payment of the debt have been frittered away.

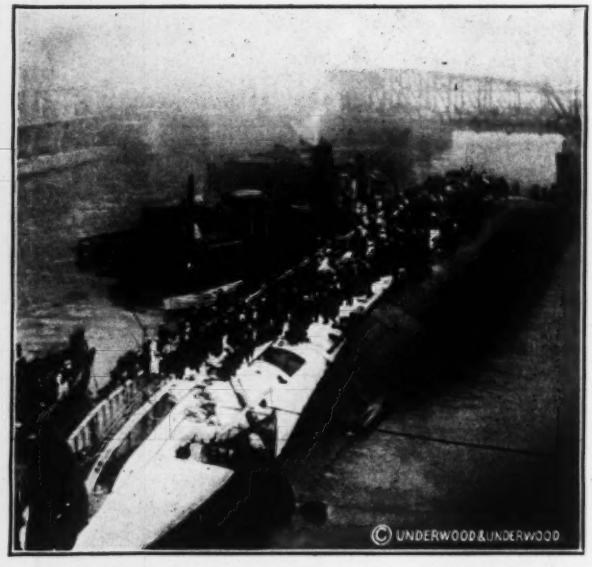
Mail-Carrying Rates.

The hotly debated question as to the fairness of the rate of compensation paid by the Government to the railways for carrying the mails is now to be tested in the courts. Seven New England railway companies brought suit in the Court of Claims on July 17 for the payment by the Government of $11,880,000, which is alleged to be the sum by which the amount received by these railways for mail transportation during the last six years falls short of fair compensation.

It is an old saying that it takes two to make a bargain. The rates of compensation for carrying the mails, however, have been for years arbitrarily fixed by the postal officials without regard to the claims of the railway companies. At present the Government pays the railways for transporting the mails on the basis of the average weight of mail carried, determining that weight by making tests every four years. No such tests have been made on the New England railways since the parcel post was established, and consequently the railways are carrying the vastly increased amount of mail due to the parcel post and receiving no greater compensation therefor.

The railways claim that the present rate of compensation is less than the actual cost to them of transporting mails, and that under the Supreme Court decision of last March in the North Dakota coal rate case railways cannot be compelled to render a service at less than cost and make up the difference by extra charges on other classes of traffic.

It will readily be understood that the contentions of the railway companies in this case are closely connected with the much-discussed matter of the rate of postage on publications and also with the general question of freight and passenger rates, since the railways maintain that they are obliged to charge more on other classes of traffic than would be the case if their postal business were remunerative.



THE CAPSIZED STEAMER "EASTLAND" IN THE CHICAGO RIVER

and that those responsible for this terrible loss of life receive due punishment.

It will be evident at once to every engineer familiar with ship construction that the primary cause of the capsizing of the "Eastland" must be sought for in the condition of the vessel's water ballast tanks. Practically all deep-draft lake and ocean vessels of the present day have water-ballast tanks in the bottom of the hull by which the draft and stability of the vessel are controlled. When a vessel is laden with heavy freight she may have ample stability with the water-ballast tanks empty; when sailing without cargo, however, it is generally necessary to fill the ballast tanks to make the vessel stable and sometimes it is necessary to add extra ballast besides to make the vessel seaworthy in all weather.

In vessels for passenger service, considerations of stability are closely connected with considerations of steadiness and consequent comfort for passengers. If a vessel has too great stability it may be so "stiff" and have such a short period of roll as to be uncomfortable for passenger service. The late Charles H. Cramp, in a paper before the American Institute of Naval Architects and Marine Engineers a few years ago, pointed out that in the endeavor to build steamers with a slow and easy roll, some of the designers of vessels for transatlantic passenger service had gone so far as to build vessels which absolutely depend upon their water ballast for stability and would capsize should their ballast tanks be empty.

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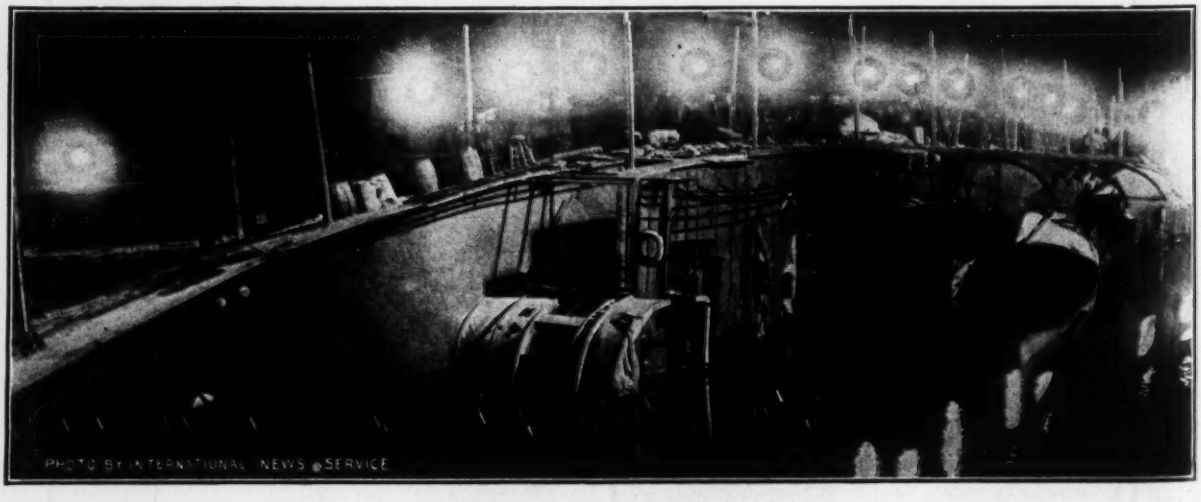

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Letters to the Editor

Pavement Foundations Over Filled-In Trenches

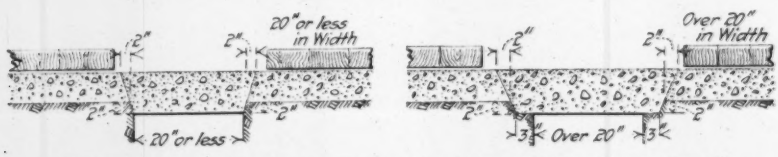
Sir—With reference to the letter of E. R. Horton, Jr., in *Engineering News*, June 17, on restoring pavement foundations over filled-in trenches, I wish to bring to the attention of your readers the practice in Detroit, Mich.

The accompanying sketch shows the method employed in restoring pavement foundations in Detroit since the beginning of the present year. This covers the relaying of foundations where trenches have been opened by any of the public-service corporations, plumbers and others.

The following regulations explain the methods employed:

All trenches and excavations shall be backfilled by such means as the Department of Public Works shall direct, depending upon the character of the excavated material and the condition of the weather, and in such a manner that all excavated material shall be replaced in the trench or excavation, without raising the grade of the roadway, making allowance for conduits, pipes, etc., placed in the excavation.

In the downtown district, where the streets and alleys are filled with pipes, conduits and underground construction, and traffic is heavy, inspectors must be called when the work of excavating is commenced, also at all times when excavations for trenches of any length are commenced. On ordinary excavations, for connecting with the water mains, or sewers, or repairing water mains or connections, the plumber or contractor must notify the permit division when ready to backfill the excavation. Material used for backfilling must be in proper condition, all wet, soggy dirt must be removed from the street and replaced with dirt of the proper consistency, or cinders. Coarse, lumpy or cloddy earth must be pulverized before being returned to the trench or excavation. From Nov. 15 to Apr. 15 all excavations in paved streets and alleys must be filled with crushed stone, thoroughly pounded down. All backfilling must be done by thorough, hard tamping, in layers not exceeding 4 in. in depth. Two able-bodied men, with heavy rammers or tamping irons to be in the excavation, for each shoveler outside. Where tunnels are made, the filling must be of concrete, consisting of 1 part portland cement to 4 parts of sand and 8 parts of broken stone or brick, satisfactory to the inspector, and under car tracks must be properly keyed up with brick or stone in addition to the concrete mixture.



METHOD OF RESTORING CONCRETE FOUNDATION OF FILLED-IN TRENCHES, DETROIT, MICH.

After backfilling is completed, the loose, broken stone must be placed on top and covered with the surface material, and left in a safe condition satisfactory to the inspector, and all the surplus dirt removed from the street, and the street properly cleared up. No backfilling whatever is to be done in the roadway or between the curb and lot-line, except in the presence of the inspector from the Department of Public Works.

No paving will be allowed on permit work unless satisfactory arrangements have been made for supervision when the permit is issued. When paving is done by contractor or corporation the trench shall be reinforced at the top by a portland-cement concrete foundation, at least 8 in. deep consisting of 1 part cement, 3 parts clean, sharp sand, and 6 parts crushed slag or hard limestone, properly mixed and tamped, the concrete to extend over the edges of the trench or excavation at least 6 in. on each side, and then properly guarded until it is set. This shall then be covered with a cushion of clean, sharp sand not exceeding 1½ in. deep. The surface paving material shall be the same as the surface of the street, the rules of the department in regard to new pavement to be followed at all times. When the surface is brick and new brick are necessary they must be of the same size or brand as those used in the street. The joints at the edges of the excavation are to be broken out and the brick properly tongued or keyed together. Whenever contractors are allowed to do the repaving in street or alley and are backfilling and concreting at the same time there must be two inspectors on the job, one to supervise backfilling and one to supervise concreting.

Besides these printed regulations the Department of Public Works issues a circular of instructions to inspectors to make sure that the regulations are followed.

GEORGE H. FENKELL,
Commissioner of Public Works.

Detroit, Mich., July 6, 1915.

Sir—It has been a rule in Kansas City, Mo., since 1911 for the city to repair all cuts made in pavements with its own forces, charging the party responsible for the cut a stated amount per square yard for repairs, and relieving the contractor having the pavement under maintenance from any further maintenance of that portion of the pavement which is actually repaired. The cut is made 6 in. wider on all sides than the actual size of the excavation, so that the concrete foundation which is replaced will have a bearing of at least 6 in. on undisturbed soil. The widening of the pavement cut is not made until the time for repairs. I believe this has done a great deal toward eliminating poorly repaired cuts.

CLARK R. MANDIGO,
Assistant City Engineer.

Kansas City, Mo., July 7, 1915.

A Garbage-Reduction Question

Sir—I noticed in your issue of May 27, 1915, an article on the success of garbage reduction in the cities of Columbus and Cleveland, Ohio.

While I do not, of course, question the facts submitted in this article, or that the plants have made a profit from the reduction of the garbage delivered to them, I wish to raise the question as to whether or not these plants are successful from an economic standpoint as parts of a general garbage-collection and disposal system. In Seattle the cost of collecting garbage—which includes not only garbage proper but all the ashes, refuse and other waste products of the city, even lawn clippings being included—is about three times the cost of disposal by the method of incineration. If, now, it is necessary to make one collection of the garbage taken to the reduction plant and another separate collection of the other forms of refuse, the extra cost of such collection may very easily over-

balance any profit which may be made in the reduction of the garbage proper. In other words, the total cost of this method may be more than the total cost of other methods in which the cost of disposal alone shows a loss instead of a profit, due to the extra cost of collection in the former method. It is, of course, praiseworthy that officials in charge of any particular plant should seek to operate it in such a manner as to give the very best possible results. Engineers, however, are interested in knowing not only the results of the operation of a particular portion of a plant but the results of the operation of the plant as a whole.

I should be glad to know if there are any data available in these cities showing the total cost of collecting and disposing of all the wastes.

A. H. Dimock,
City Engineer.

Seattle, Wash., June 25, 1915.

Patentees' Royalties Justified

Sir—In *Engineering News*, June 3, 1915, p. 1088, is a letter signed "Contractor," referring to the contract for a patented retaining wall let at Tiffin, Ohio, Apr. 20, 1915.

"Contractor" reasons badly in saying that the patentee who himself bid on his own patented plans had an advantage (by reason of specifying 63c. per cu.yd. as royalty) over others who bid on those same plans. He had specified that amount, approximately 10 per cent. of the contract price, for all bidders as his recompense for the use of his patent. In all but one or two states, court decisions have held that public lettings on patented devices are legal and fair when the amount of royalty is specified so that bidders may secure the right to use the patent. In some of the states this principle as established by the courts has been enacted into statutes, and it is desirable legislation for the reason that so many contractors are ignorant of the principles underlying fair competition in patented devices at public lettings.

Assuming that all the bidders figure the same cost and profit, then the patentee would file the same bid as the others if he included the specified royalty for himself. Since he can collect this specified amount from the successful bidder, he cannot reduce that amount in his own bid. If his bid proves to be lower than the others it is because he has cut his profits, not his royalty, for that royalty is available to him from any other successful bidder. And the patentee is thus on an equality with all other bidders. This principle applies regardless of how worthless the patent may be or how high the specified royalty, for how can the patentee cut his royalty in his own bid when that very amount is tendered him by his competitors? And if he could have deducted any part of the 63c. specified, who but the taxpayers would be the immediate benefactors? What the patentee did at Tiffin was merely to comply with court decisions.

Contractors and civil engineers very generally refuse to recognize the merits of the patent system. This is the more surprising because patents have much to do with our industrial progress, and so have contractors and engineers. In fact, industrial progress has been so extended by science that today the only man that can hope successfully to invent is the scientist. Hence, logically, the civil engineer should be the inventor of today. But he is not; and instead of blaming himself he excuses his own barrenness by charging the Patent Office with carelessness, a charge which is often heard but has never yet been substantiated except by a few exceptions, whose prominence is a good indication that carefulness is the rule.

"Contractor's" article contains another striking illustration of the harm that is done the community by the attitude of contractors and civil engineers toward patents. The patentee quoted the city $1200 for the use of his patented device. This was refused. Then a royalty of 63c. per cu.yd. was specified to be included in the bid.[1] The 4100 cu.yd. contained in the wall thus cost the city $2583, unless the contractor took chances on litigation, which he would hardly do, for this patent is held valid in that circuit. Notwithstanding this added royalty, the patented wall was bid one-third lower than the plain wall, with a saving of $15,000 to the city. Why did the city refuse to pay $1200 and then pay $2583 instead for what was so obviously of advantage? Probably because of criticisms aroused by contractors and engineers, of which the contribution to your journal by "Contractor" is an excellent example. Thus the continual "knocking" that is done against engineering patents not only is an obstruction to progress in the arts but also increases taxes.

Daniel B. Luten.

Indianapolis, July 10, 1915.

NOTES AND QUERIES

New York Standard Gate Valves—In "Engineering News," May 27, 1915, p. 1018, in an article on the standard waterworks valves of New York City, the price of a 16-in. valve was included, although a 16-in. valve is not one of the five standard sizes now in use. It was an oversight to include this in the list as it naturally raises the question of the city using other than the standard sizes, although the article states specifically that it does not. W. W. Brush, Deputy Chief Engineer of the Department of Water Supply, Gas and Electricity, states that the price for a 16-in. valve was quoted, as the department had purchased a few of these valves to be set on existing 16-in. lines where the conditions did not make it advisable to use the standard 12-in. valve. No price is given for 48-in. valves as no valves have been purchased under the department design of this size. The price of the 48-in. valve of the department design is problematical, due to the high cost of the pattern that would have to be charged if such a valve were now to be purchased. If a number of these valves were to be used so that the pattern cost would be negligible an approximate estimate of cost is $1200 each.

Oiled Roads Destructive of Fish Life—Near Southampton, L. I., there is a large fresh-water lake, separated from the ocean by sand dunes and fed by springs and surface water. The lake is about a mile long and in some places half a mile in width. It was formerly well stocked with fish. The water draining from the oiled roads in the vicinity, however, has killed all the fish and the question has arisen how this destruction of fish life may be prevented so that the lake may be restocked. It is estimated that there are at least ten miles of oiled roads, the drainage from which sooner or later reaches the lake. It has been proposed to build a large concrete tank in which the grit and sediment washed from the roadways would settle and could be removed at intervals, while the oil would rise to the surface and could be drawn off. The question is whether water drawn from between the top and bottom and discharged into the lake would be free of matter injurious to fish. It is thought possible that some of the tarry acids washed from the roads are soluble in water and that this plan therefore might not be successful. The engineer in charge of the work, A. L. Stevenson, 101 Park Ave,. Southampton, L. I., would be glad of information from any engineer having had charge of similar work.

[1] This is not precisely correct. The city notified prospective bidders that they would have to protect the city against patent claims. A patentee of a retaining wall then announced that he would charge 63c. per cu.yd. for all concrete put in on the proposed design.—Editor.

Ohio River Bridge for the C., B. & Q. R. R.

SYNOPSIS—This great bridge, now under construction, is notable for the use of silicon steel and for having the largest plain truss span ever built (723 ft.). It is a double-track bridge, designed for exceptionally heavy live loads. The article outlines the requirements of the steel and the loading.

The new Ohio River bridge at Metropolis, Ill., now under construction by the Chicago, Burlington & Quincy R.R., will have as three special features: The longest fixed truss span yet built, 722 ft. 11 in. c. to c. of piers (with four others of about 556 ft.); the use of silicon steel for the truss members; and provision for very heavy loading. A general drawing and description of the bridge were given in *Engineering News*, July 30, 1914, and the drawing is reproduced herewith. Preliminary work was started about that time, but was stopped very soon after, owing to financial conditions resulting from the war in Europe. The railway connections are nearly completed, however, and a car ferry is being established across the river. In June, 1915, it was decided to proceed with the bridge, and work is now in progress on the pier foundations and the fabrication of the steel.

The bridge was designed by C. H. Cartlidge, Bridge Engineer of the Chicago, Burlington & Quincy R.R. (and Chief Engineer of the auxiliary Paducah & Illinois R.R., which is building the bridge and its connections). All detail drawings were prepared in the Bridge Department, these being submitted for the approval of Ralph Modjeski, consulting engineer for the bridge. The Union Bridge & Construction Co., of Kansas City, Mo., has the contract for the substructure. The American Bridge Co. has the contract for the fabrication and erection of the superstructure, and is building the steelwork at its shops at Gary, Ind.

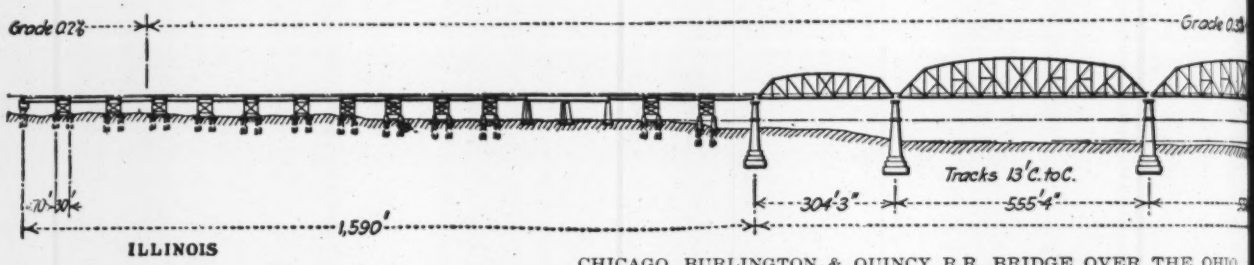
The structural design of the main trusses is very heavy. The top chords and end posts are of H-section, with two built-up channels for the sides and a continuous diaphragm. The top and bottom lacing is of angles and bars respectively. Some of the plates will have a thickness as great as 1¾ in. The largest pins are 15-in. diameter. The top chords are riveted throughout, and very large gussets are used.

An ordinary open-floor deck will be used on the truss spans, while the approach viaduct will have a solid deck of reinforced-concrete slabs for ballasted track.

The river piers will be of reinforced concrete, sunk by the pneumatic process to about 85 ft. below high water, and resting in a deep bed of gravel. Concrete piles will be used under the footings of the viaduct approaches in some cases, while in others these foundations will be on a bed of cemented gravel.

Character of the Steel Used

In the trusses, all main members except the eye-bars will be of silicon steel, the eye-bars and pins of nickel steel and all other material (including lacing) of medium steel. The material in the deck plate-girders and towers of the viaduct approaches will be of medium steel. This is the first use of silicon steel for heavy bridge work in this country. The purpose of using it is to combine high strength with a low increase of weight, in view of the high loading which the bridge is designed to carry. The cost is not very much in excess of that of medium steel and less than for nickel steel. It requires rather more care in the mills and the shops and will be rather slower for drilling. All the material will be openhearth steel.



CHICAGO, BURLINGTON & QUINCY R.R. BRIDGE OVER THE OHIO
C. H. Cartlidge, Chief Engineer.

All rivet holes in silicon steel (which will be ⅞ to 1½ in. thick) will be drilled from the solid to $\frac{1}{16}$ in. excess diameter, and this will apply also to medium structural steel 1 in. thick and upward. For medium steel less than 1 in. thick the holes will be subpunched and reamed. Any sheared plates or shapes in the trusses will have the edges planed or faced.

The rivet steel is similar in composition to the structural steel except that its sulphur limit is 0.04% and its maximum ultimate strength is 55,000 lb. with a yield point 55% of this. In unit stress the shear and bearing are 11,200 and 19,000 lb. respectively for shop rivets, and 9400 and 16,000 lb. for field rivets.

The nickel-steel eye-bars tested to destruction must show a minimum of 80,000 lb. ultimate tensile strength and 48,000 lb. yield point by drop of beam, also 10% elongation in 18 ft., with 25% reduction in area and a silky fracture. Full-sized material for these eye-bars, 1 in. thick and over, must bend cold 180° around a pin three times the thickness of the bar, without a sign of fracture.

The requirements for the three grades of steel are given in the accompanying table, and in regard to the chemical requirements it is specified that the material must be uniform or of approximately the same composition throughout.

CHARACTER OF STEEL FOR THE OHIO RIVER BRIDGE OF THE CHICAGO, BURLINGTON & QUINCY R.R.

	Structural Steel	Nickel Steel	Silicon Steel
Phosphorus, max. (basic)	0.05%	0.04%	0.04%
Phosphorus, max. (acid)	0.08%	0.05%	0.06%
Sulphur, max.	0.05%	0.04%	0.05%
Nickel, min.	.....	3.25%	.....
Carbon, max.	.....	0.45%	0.40%
Manganese, max.	.....	0.80%	1.00%
Manganese, min.	.....	0.50%	.....
Silicon, min.	.....	.....	0.25%
Ult. strength, min.	55,000	95,000	80,000
Ult. strength, max.	65,000	110,000	95,000
Yield point	30,000	55,000	45,000
	1,500,000	1,500,000	1,600,000
Elongation in 8 in.*	Ult.	Ult.	Ult.
Reduction of area, min.†	40%	25%	35%
Fracture desired	silky	silky	silky
Cold bend without fracture: ¾-in. thick	flat	d = 2t	d = t
Cold bend without fracture: ¾ to 1½ in.	d = t	d = 3t	d = 1½t
Cold bend without fracture: over 1½ in.	d = 1½t	d = 3t	d = 2½t

*Elongation in 8 in.—For silicon and nickel steel a deduction of 1% will be allowed from the specified elongation percentage for each increase of ¼ in. (or fraction thereof) in thickness above 1 in., but the elongation must not be less than 14%. In structural steel, a similar deduction will be allowed for a similar increase in thickness above ¾ in. †Reduction of Area—For material ¾ in. and less in thickness a reduction of 1% will be allowed for each increase of ⅛ in. above ¾ in., with a minimum allowable reduction of 24% for silicon steel.

Load and Stress Requirements

The very high live load for which the bridge is designed is a striking feature. The live load assumed for the truss design is as follows: On the near track, two engines of the 2:8:0 class of Cooper's E-90 loading, followed by a train load of 7500 lb. per ft. of track; on the far track, the train load only. For hangers and subdiagonals, however, two engines are assumed for each track. For the approaches the assumed live load for each track consists of two Cooper's E-90 engines followed by the train load mentioned.

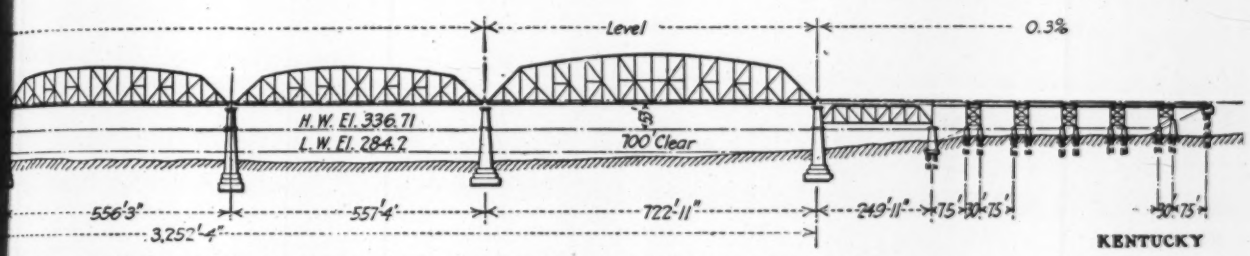
It was the engineer's opinion that the bridge should be designed and built for the ultimate probable load. The reason was that, with spans of varying lengths and with loads nearly equal to (or perhaps a little above) the present existing equipment, the resultant design would not provide the same percentage of strength after the loads should have reached the maximum allowable. This is due, of course, to the fact that the dead load of a long span is a much larger proportion of the total load on that span than it is of the total load on a shorter span. By proportioning the spans for the ultimate probable load a uniform relation between the dead and live loads is maintained, and when the loads for which the bridge is designed are finally realized the spans will be stressed throughout to the amounts decided upon in designing.

This applies particularly to the engines, and it was considered that 45-ton axle loads might reasonably be regarded as a maximum. For the train load, however, the same figure was used as for previous large bridges, since there is little indication of increase in weight per foot of track, in spite of heavier car loads and axle loads. Thus solid coal trains on the Chicago, Burlington & Quincy R.R. do not run higher than 4000 lb. per ft. of track, and with merchandise freight in solid trains it is only about 3000 lb. per ft. of track.

The dead load of track and deck is taken at 500 lb. per ft. of each track for the truss spans (with open floor construction), and 3350 lb. per ft. of each track for the ballasted concrete deck on the approaches.

Impact is provided for in the trusses by adding 10% to the live-load stress in all members, except that for floor hangers and subdiagonals 50% of the live load for one track is to be added on the near track. The 10% addition is increased to 20% for the 246-ft. deck-truss and 300-ft. through-truss spans. For impact on the approaches 22,500 lb. is added to each of the four drivers on one rail of each track for the engine which will cause the greatest stress.

Wind load on the trusses is taken at 30 lb. per sq.ft. of area of train and floor in elevation, for 20 ft. in height; also the same load for the area of truss members of two trusses not covered by this height. All are treated as moving loads. For the top laterals on through spans and the bottom laterals on the deck span, the wind load is taken at 50 lb. per sq.ft. on the exposed area of truss members for two trusses. This also is treated as moving load.



RIVER AT METROPOLIS, ILL., CONTAINING THE LONGEST PLAIN TRUSS IN EXISTENCE
Ralph Modjeski, Consulting Engineer.

For the approaches the viaduct towers are designed for a wind load of 50 lb. per sq.ft. on 1½ times the vertical projection of the structure unloaded; or 30 lb. on the same surface plus 400 lb. per lin.ft. of structure applied 7 ft. above the rail for assumed wind force on a train when the structure is either fully loaded or loaded on either track with empty cars assumed to weigh 1200 lb. per lin.ft., whichever gives the larger strain.

The traction, or longitudinal force, is provided for in the viaduct towers by an allowance for a longitudinal force of 20% of the live load applied at the top of the rail, on one track only. Only a short section of the land end of one approach is on a curve, but the specifications provide an allowance for centrifugal force of the live load applied at the top of the high rail. This force is considered as live load and is derived from the formula $60 - 2\frac{1}{2} D$, in which 60 is the speed in miles per hour and D the degree of curve.

The following unit stresses are allowed for the total dead load, live load and wind load:

	Silicon Steel	Medium Steel	Nickel Steel
Tension and compression*	30,000	20,000	35,000†
Tension and compression for wind	30,000	24,000	35,000
Shear, webs	.....	11,200	.....
Shear, pins	.....	.....	20,900
Bearing, pins	30,000	20,000	35,000
Bending, plates and shapes	30,000	20,000	.....
Bending, pins	.....	.....	35,000

*Compression as reduced by Gordon's formula. †Tension only for nickel steel (eye-bars).

For members having alternate stress, the gross and net sections required for compression and tension stresses respectively are determined and half the smaller section is added to the larger section. The net sections and gross sections are used where tension and compression respectively are the governing stresses. Pin plates are proportioned for the sum of both stresses, except where governed by the section required at pin holes. Members subject to both axial and bending stresses are so proportioned that the combined fiber stresses will not exceed the allowed axial stress.

Garbage-Disposal Propositions at San Francisco

The City Engineer of San Francisco, M. M. O'Shaughnessy, made a report to the Board of Supervisors of the city, on July 2, summarizing various propositions which have been made to the city for the disposal of its garbage and refuse. E. G. Borden offered to erect, at his own cost, a 50-ton experimental hydro-incinerator under the Heslewood patents. The incinerator may be briefly described as a modified gas producer. Mr. Borden claimed that the plant would produce low-grade fuel gas and clinker, both salable articles, and estimated the cost of a 50-ton plant at $8650. After the plant's success had been demonstrated he would sell it to the city at cost, the city to pay in addition $50,000 for the use of the patents covering its construction, with the right to erect such additional plants as it might require.

Collins & Pellett offered to install garbage incinerators with a capacity of 512 tons per day of 24 hr. for $340,000, guaranteeing them to be odorless and practically smokeless.

Fred T. Smith offered to reconstruct the old Thackeray incinerator plant belonging to the city for $255,000, so that it would have a capacity of 720 tons per day of 16 hr. As an alternate proposition he would rebuild the plant for $85,000 and operate it for ten years for 60c. per ton of garbage delivered, the city to guarantee an average of 500 tons or more per day. He also offered to construct a new plant of 720 tons' capacity per day of 16 hr. for $510,000, or for $175,000 with a ten-year operating contract. This proposition was accompanied by detailed specifications as to the operation of the incinerator without dust, smoke, odor or other nuisance.

T. J. Lacey, of San Antonio, Tex., offered a scheme for utilizing garbage in the manufacture of fuel briquettes. J. E. Briggs offered the De Carie incinerator. H. B. Harris, of Nashville, Tenn., offered a garbage incinerator of his own design. Charles Turner, Vice-President and General Manager of the San Francisco Disposal Co., proposed a separate collection of garbage and refuse, the latter to be disposed of in the Islais Creek incinerator and the former to be treated by the reduction method in a plant that his company has built and would enlarge to provide sufficient capacity for the city. J. M. Hirsch, of Chicago, offered a chemical treatment for garbage claimed to produce valuable byproducts.

Richard Schmidt asked for a 35-year franchise to dispose of all the garbage, refuse and waste matter collected in the city by dumping it on waste land and covering it with layers of earth, at a price of 60c. per ton delivered, including that delivered by private scavengers as well as by the city. For such a franchise he offered to pay to the city the sum of 3c. per ton during the first year of operation and 4c. per ton thereafter.

Mr. O'Shaughnessy has investigated the method of garbage disposal proposed by Mr. Schmidt, which is in use at Seattle, Wash., and submitted statements from the President of the Board of Health, the City Engineer and the head of the refuse-disposal department of Seattle, expressing satisfaction with this method as used in that city. Mr. O'Shaughnessy recommended that before entering into any negotiations for the disposal of San Francisco refuse by incineration or reduction, a thorough trial should be made of the method proposed by Mr. Schmidt.

Water Pollution Enjoined at Niagara Falls

The Supreme Court of New York issued an injunction on July 5 restraining the City of Niagara Falls from discharging the effluent of the municipal water-works filter plant in such a manner as to pollute the water taken from the Niagara River by the Western New York Water Co. The city and the company are competitors in furnishing water to the people of Niagara Falls. The municipal water-works, located about two miles above the city, takes water from the river through an intake extending some 2000 ft. beyond the shore, carrying it to a filter plant built in 1912. There is discharged into the river from this filter plant annually about 550 tons of suspended matter from the sedimentation basins, an equal amount of solids in effluents from the filter beds, and about 300 tons of chemicals used in treating the water at the filtration plant.

About two miles down the river from this municipal water-works filter plant is the intake of the Niagara Falls Power Co., from which the Western New York Water Co. draws its supply for domestic use. The water company claimed that the discharge from the municipal filter plant above materially increased the amount of suspended matter in its water supply, as well as the percentage of bacteria, thereby subjecting the company to greater expense in filtering the supply before delivering it to consumers. The river, however, carries a large amount of pollution anyway, and the attorneys for the city contended that, this being the case, it was immaterial that the effluent from the municipal filter beds was discharged into the river. The attorneys for the company, however, showed that there was a trunk sewer near the municipal filtration plant into which the effluent from that plant could be discharged as conveniently as into the river, and with little more expense, and which would deliver the effluent below the falls.

Under these conditions the court decided that it was reasonable to require the city to discharge the effluent from its filtration plant into the sewer instead of into the river itself. Issuance of the injunction was delayed for six months to give the city time to make this change. The case was notable for the conflicting evidence of the chemists testifying on opposite sides of the case. The court decided in favor of the testimony presented by the plaintiff on the ground that the tests made by its chemists were the more reliable, in that they tested the acidity of the gelatine used in their examinations. The experts for the plaintiff were H. F. Huy, of Buffalo, and James M. Caird and F. J. Longley, of Troy.

Concrete-Paved Automobile Racetrack

A two-mile automobile speedway or racetrack 60 ft. wide, with a 6-in. concrete pavement, is under construction near Fort Snelling, in the southwestern part of St. Paul, Minn. Races are scheduled for Sept. 4, and every effort is being made to rush the work.

A sketch plan of the track is shown in Fig. 1. It consists essentially of two short tangents (549.6 ft.) and two circular arcs of 1245.89 ft. radius, each 3130.4 ft. long and each connected to the tangents by easement spirals

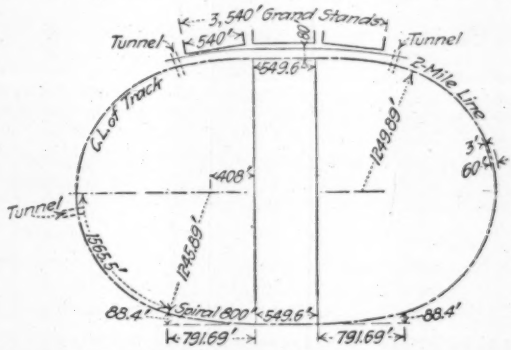


FIG. 1. SKETCH PLAN OF TWIN CITIES MOTOR SPEEDWAY

800 ft. long. The track proper is 60 ft. wide, but in front of the grandstands the width is 80 ft.

The curves are banked by an earth fill 13 ft. high. The concrete pavement on the higher half of the fill is reinforced. Fig. 2 is a cross-section on the 800-ft. easement curve, and the accompanying table gives the elevations of the four governing points on the spiral cross-sections.

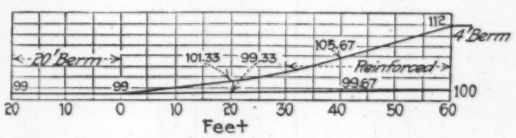


FIG. 2. CROSS-SECTION ON CURVES

The inner edge of the track is a level around the entire circuit.

The original surface was fairly level, of a sandy soil. The fills are made in 2-ft. layers thoroughly rolled damp with a 10-ton roller. As an additional safeguard against

TABLE SHOWING ELEVATIONS AT THE VARIOUS STATIONS ON SPIRAL

Station	0-Ft. Point	20-Ft. Point	40-Ft. Point	60-Ft. Point
15 to 15	99.0	101.33	105.67	112.00
16 and 37	99.0	101.11	105.00	110.67
17 and 36	99.0	100.89	104.33	109.33
18 and 35	99.0	100.67	103.67	108.00
19 and 34	99.0	100.44	103.00	106.67
20 and 33	99.0	100.22	102.33	105.33
21 and 32	99.0	100.00	101.67	104.00
22 and 31	99.0	99.77	101.00	102.67
23 and 30	99.0	99.55	100.33	101.33
24 to 29	99.0	99.33	99.67	100.00
38 to 38	99.0	101.33	105.67	112.00

settlement, the greater part of the pavement slab on the fill is reinforced, as noted.

The concrete pavement is being constructed with regulation pavement type of portable concrete mixers. Expansion joints are placed every 50 ft. diagonally across the track (at 30 deg. to the inner and outer edges). A continuous longitudinal joint extends through the length of the pavement. Concrete tunnels 60 ft. wide will give access to the interior of the speedway area.

This track is being built by the Twin City Motor Speedway Co., for which O. H. Case is chief engineer. Barnard & Frederick, of Minneapolis, are contractors for the 189,000 cu.yd. of earthwork. Hanlon & Akes, Sioux City, are contractors for the concrete, and J. F. Sperry, St. Paul, is contractor for the reinforced-concrete grandstands, which are designed to seat 75,000 spectators.

Repair of Defects in Reinforced Concrete

A comprehensive review of methods which have been employed to repair cracked elements of reinforced-concrete structures was given some time ago by Prof. S. Müller before the Deutsche Beton Verein. His account was based on extensive experience in carrying out such repair or reinforcement, chiefly of floor slabs and beams.

The defects in question include weakness of the slab due to inadequate thickness for proper service as compression element of the T-beam; separation of slab from beam; and diagonal or horizontal cracking of the beam web. Vertical cracking, which sometimes occurs, is due to shrinkage and is not considered dangerous.

Beams

In the earliest repair jobs on defective beams the plan adopted was to strengthen the floor by adding supplementary beams or by setting a column under the middle of the weak beam. In some cases a pair of I-beams has been placed alongside the weak concrete beam; but this is generally uneconomical, because the uncertainty of the load distribution makes it necessary to proportion the steel beams to take the entire load. Diagonal cracking, indicating a deficiency of shear reinforcement, has been repaired by placing vertical or inclined steel stirrups around the web of the beam and screwing them up tight. This method did not always prove satisfactory, however, especially in structures subject to vibration. The best method of repair yet developed is to place cramps over the crack, doweled into the sound concrete on both sides, and incase them in a shell of concrete, for which space may be made by chipping off the original concrete for a depth of an inch or so.

Slabs

Where a slab requires to be reinforced, to provide additional compression area for T-beam action, a good bond of the new top layer has generally been obtained. Thorough cleaning of the surface is first necessary, followed by roughening the old concrete by picking or otherwise; after wetting the surface, a 2-in. layer of concrete well reinforced by a strong mesh is placed. The mesh should be tied to the reinforcement of the old concrete by steel-tie members.

Where slab and beam have separated the scheme has been employed of adding such a reinforced upper layer on top of the slab and tying this down through holes in the slab to cross-bars passed through holes in the stem of the beam.

It has been found that painstaking work is necessary for success in repairing reinforced-concrete structures. With care, however, the patches or new elements can be made structurally integral with the old parts. Satisfactory appearance can be secured. In the best work the patches are practically invisible.

News of the Engineering World

Denver Plans Water-Works

The Public Utilities Commission of the city and county of Denver, Colo. on July 19 entered into a contract with Van Sant-Houghton Co., engineers and contractors of Denver and San Francisco, for making surveys, plans, specifications, and a detailed report preliminary to asking for bids on the construction of a complete new water-works plant for the city. All questions relating to water-supply, location, cost, etc., will be considered. The work will commence at once and will require several months for completion.

Imperial Valley Earthquake Tests Engineering Works

[CONTRIBUTED]

A few brief notes on the earthquake of June 22, 1915, in the Imperial Valley, California and Mexico, were given in *Engineering News*, July 1, p. 46. A more detailed examination has now been made than was then possible and shows that the early reports of damage were exaggerated; the shock, however, was severe.

Only those buildings suffered which were of inferior construction. Brick and concrete structures having any reinforcement at all were practically untouched. Frame buildings sustained very little damage. On reinforced-concrete work only the surface decorations were injured, except where there was some fire damage. In several structures hair-cracks were found penetrating as far as the reinforcement, but stopping there. Most of the damage was due to lack of proper ties and bonding in the cheap brickwork, which had been permitted with the rapid growth of the cities and in the absence of building restrictions.

The Masonic Temple in Calexico, Calif., a well-tied and anchored two-story brick building, was uninjured, though reported in the newspapers as a wreck. Calexico had two fires, but they were controlled, since the damage to water mains was slight. The loss in Calexico is estimated at $100,000 maximum; at El Centro it may rise to $500,000. Throughout the valley there was little injury to city improvements.

A most interesting result is the good showing made in river-protection and irrigation works. There was no injury of consequence at the old breaks of the Colorado River. On the Imperial Valley irrigation lines some of the embankments developed shallow longitudinal settlement cracks; the puddling around some structures cracked away, but not enough to cause serious leakage. In places timber work was found out of line.

The main canal was carrying 4500 sec.-ft. at the time of the shock, but it was not found necessary to diminish the flow. Along some of the minor canals which were running near their capacity water was shaken over the levee tops; but the disturbances did not continue long enough to cause serious breaks. The New River flume, a $90,000 project just completed, is intact. This flume is carried on a timber trestle, in some places elevated 50 ft. on pile bents driven 35 ft.

The net result of this earthquake probably will be beneficial. Only a few days elapsed before the towns began passing much needed ordinances regulating building construction. As a result of this severe test of the engineering works in the valley, the confidence of the people in their safety and value has been greatly strengthened.

TYPICAL EXAMPLES OF EARTHQUAKE DESTRUCTION IN CALEXICO, CALIF.

The Naval Inventions Board

The invitation from Secretary Daniels to the principal national engineering societies to nominate representatives on the proposed Naval Inventions Board appears to have been due, partly at least, to the initiative of Frank J. Sprague. Mr. Sprague was graduated from Annapolis and spent five years in the navy after graduation, resigning in 1884 to engage in the pioneer work of electrical development. The following is quoted from Mr. Sprague's letter to Secretary Daniels:

Permit me to congratulate you upon having decided to create a Board of Invention and Development, to be aided by an advisory board of civilian engineers, a project the necessity for which I have long had in mind. The great necessity for and possible utility of such an organization cannot be better set forth than as stated in your published letter to Mr. Edison, for it is unquestionably true that the United States is woefully deficient not alone in the actual preparation for defense but in the knowledge of many things essential to such defense.

On account of the limitations in authority and expenditure, and the routine hitherto pertaining to Government matters, it has been almost impossible for men with meritorious inventions to have them properly considered, or even if considered to have them developed. It has, therefore, fallen upon many in civil life to help develop and finance inventions of this character. Since my own resignation from the naval

service and the taking up of industrial development in the electrical field, I have had many matters presented to me which would have warranted thorough governmental investigation, and sometimes I have lent a hand.

It seems to me that provision for investigation and possible experimental equipment should take into account the needs of both services, for not only are many inventions equally valuable for the navy and the army, but researches can be carried out jointly far more effectively and with material saving in expense.

I am, perhaps, not warranted in offering advice, but on account of the vital importance of the proposed undertaking, and because such measure of professional success as has come to me is largely due to my naval training, I will venture some suggestions:

First. In view of the confidential character of most of the communications to the board and the possible attitude of "hyphenates" in critical times, your advisers, whether men of purely civil education or men in civil life who have had the advantages of West Point or Annapolis training, should be dyed-in-the-wool Americans from birth up.

Second. With sane inventors there should be included men who are also engineers who deal with constructive problems, and who can pass upon the practicability and value of inventions in the same manner as they now do for great commercial organizations—an essential if the highest efficiency is to be attained.

With regard to this latter suggestion it might be well to call into preliminary consultation the presidents of the various national, and perhaps some other, engineering societies before making a final selection. These include in their membership most of the prominent engineers and inventors in the country, the men on whose work much of our great industrial development has been founded.

As a past-president of three of these societies, I feel warranted in expressing the opinion that if the present presidents were requested by you to discuss the formation and membership of the proposed advisory board, and were asked to submit a number of names whom they could recommend for consideration as members, they would be glad to comply with your request.

Another East River Tunnel

A project for a tunnel under the East River from 59th St. in the Borough of Manhattan to Long Island City was approved by the Board of Estimate and Apportionment of New York City on July 21. It has hitherto been planned to carry subway trains over the East River from Manhattan to Long Island City by means of the Queensboro cantilever bridge, which is between 59th and 60th St. The original plans for this bridge, however, did not provide adequate strength for the operation of the subway trains, except by spacing them some distance apart. This matter was fully discussed in the report on the design of the bridge made at the time of its completion by Boller & Hodge and published in *Engineering News*. It has been estimated, however, that at an expense of about $2,500,000 the bridge could be strengthened so that it would be safe for the use of subway trains.

An offer was made to the city last spring by Michael J. Degnon, president of the Degnon Contracting Co., which has the contract for subway work at 59th St., offering to build a tunnel under the East River for the city at that point at an expense not to exceed $4,500,000; or, if both tracks of the tunnel were located under 60th St. instead of one through 59th St. and one through 60th St., as originally planned, for a maximum of $4,000,000.

It will be necessary for the city to prepare plans and specifications and advertise for bids for the tunnel construction; but Mr. Degnon's agreement to submit a bid not exceeding that amount in case such plans are adopted furnishes a guarantee to the city as to the maximum cost. Including interest charges on each of the works while under construction, the cost of dredging in connection with the tunnel and some other items, it is estimated that the additional cost of the tunnel plan over the bridge plan would be about $2,500,000.

The Public Service Commission, which had the matter under consideration in the spring, decided in favor of the bridge plan. The Board of Estimate, however, at the meeting referred to, reached a contrary decision and adopted resolutions notifying the Public Service Commission that it was ready to provide funds for proceeding with the tunnel plan. The arguments which have influenced the Board of Estimate in its decision in favor of the tunnel plan are that in view of the rapid increase in motor trucks and the rapid growth of Long Island City as a manufacturing district, as well as in population, the full capacity of the Queensboro Bridge will eventually be needed for vehicle traffic, and no more railway tracks should be placed upon the structure.

There is, further, a considerable operating economy in hauling subway trains through a tunnel under the river rather than across a bridge over it. The subway tracks are normally 20 to 30 ft. below the street level; and in order to rise from this level and pass over a bridge about 150 ft. above the water, power must be expended to lift the train the whole distance, and grades of over 5% must be used. In passing through a tunnel under the river, on the other hand, the vertical descent is very much less than the rise required to cross the bridge. Furthermore, the momentum gained by the train in running down the grade of one approach accelerates it without any consumption of power, and in ascending the grade on the opposite side the opposing grade serves to slacken the train's speed as it approaches the station.

Sanitary Engineering Bureau for Illinois

The Illinois Sanitary Engineering Bureau has been organized recently as a department of the State Board of Health, appropriations amounting to $11,550 per annum having been made to the board at the last session of the state legislature. This is the result of an active campaign by Dr. G. St. Clair Drake, secretary and executive officer of the board. The items provided in the bill are as follows, all except the last being annual amounts: Chief sanitary engineer, $3600; two assistant engineers, $1200 each; stenographer and clerk, $900; office and laboratory supplies, postage and expenses, $1750; traveling expenses, $1500; fixtures and other equipment for office and laboratory, $1400.

Plans for work which will be undertaken at the outset have been discussed only briefly, but the board has already received a large number of requests for advice and counsel on matters pertaining to sewer installations, garbage-disposal plants and sanitary investigation of school houses, the latter made necessary by a law recently enacted. It is desired also to formulate rules and regulations to govern the sanitary conduct of construction camps, summer resorts, picnic groves, and state and county fair grounds.

For the purpose of promoting the immediate organization of this new bureau, the State Board of Health has the services of Paul Hansen, Engineer of the State Water Survey, who will take a leave of absence from his present position long enough at least to develop this new service of the board. Eventually all positions in the bureau will be filled through competitive test conducted by the Civil Service Commission.

Progress on Hell Gate Bridge, New York City

The accompanying views show the Hell Gate bridge of the New York Connecting R.R. at one of its most interesting stages of construction. As these photographs were taken a very particular operation was being carried out by the erection forces of the American Bridge Co.

The two halves of the arch are erected as cantilevers, with temporary backstays holding them up, as shown in the sketch in *Engineering News*, Jan. 8, 1914, p. 62, and Apr. 8, 1915, p. 698. The east or Astoria end of the bridge has been erected for seven panels—about 300 ft.—with a backstay hitched to the end pin of the top chord (see *Engineering News*, Apr. 8, 1915, pp. 698 and 699).

For the rest of the half-arch a backstay leading over a higher point of support and hitched to the fifth panel point of the top chord is to do duty. The second backstay has been completed during the last few weeks, and

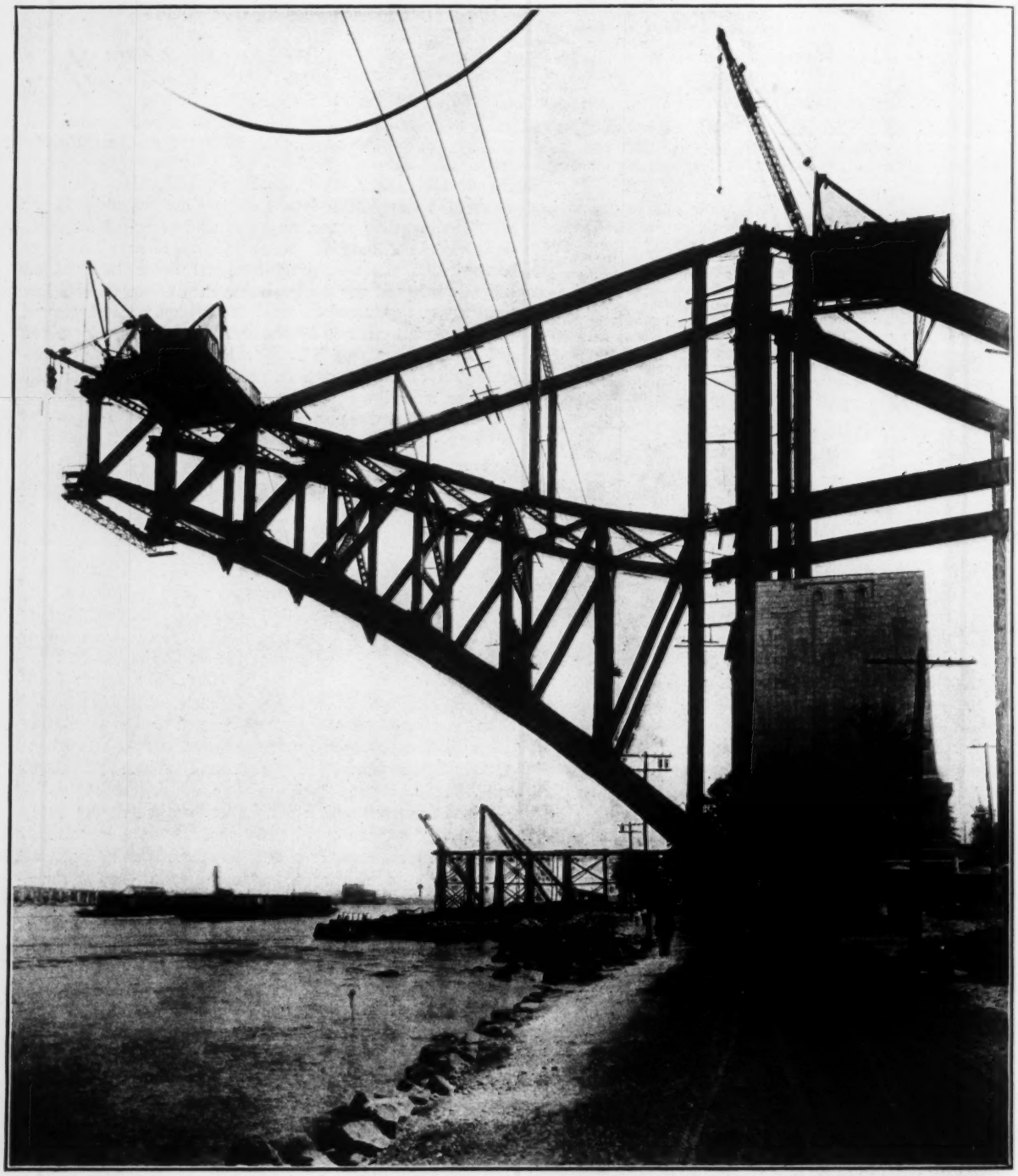


FIG. 1. EAST OR LONG ISLAND END OF HELL GATE ARCH ACROSS THE EAST RIVER AT NEW YORK CITY

on July 22 and 23 the load was transferred to it from the first backstay. At the time the view in Fig. 1 was taken a gang of men was at work driving out the last pin of the lower backstay.

On top of the new backstay supports and under the saddles of the second backstay were set two enormous hydraulic jacks, each of 2400 tons lifting capacity. The jacks were operated to lift the saddles until the short lengths of eye-bar which made the hitch of the first backstay were slacked off. As computed beforehand, this required a jack lift of about 20 in.

The eye-bars were watched, and they became slack at a lift within a fraction of an inch of the computed amount. Then the eye-bar pins were driven out, the upper saddles being blocked in position with plate blocking. There were eight eye-bars to each truss, four inside the chord and four outside, but both pins came out with no special difficulty.

The jacks will be used again in making the closure of the arch at the center.

The west or Ward's Island end of the arch is now three panels out, and the upper backstay is being erected. This end has reached about the same stage of construction as the east end had when the views in *Engineering News*, Apr. 8, 1915, were published. Work will be pushed until this end is as far along as the east end, and then both will be built forward to the center, and the two halves of the arch lowered to connection. The present season will see the arch closed at the crown. The viaduct approach from the west is nearing completion as shown in Fig. 3.

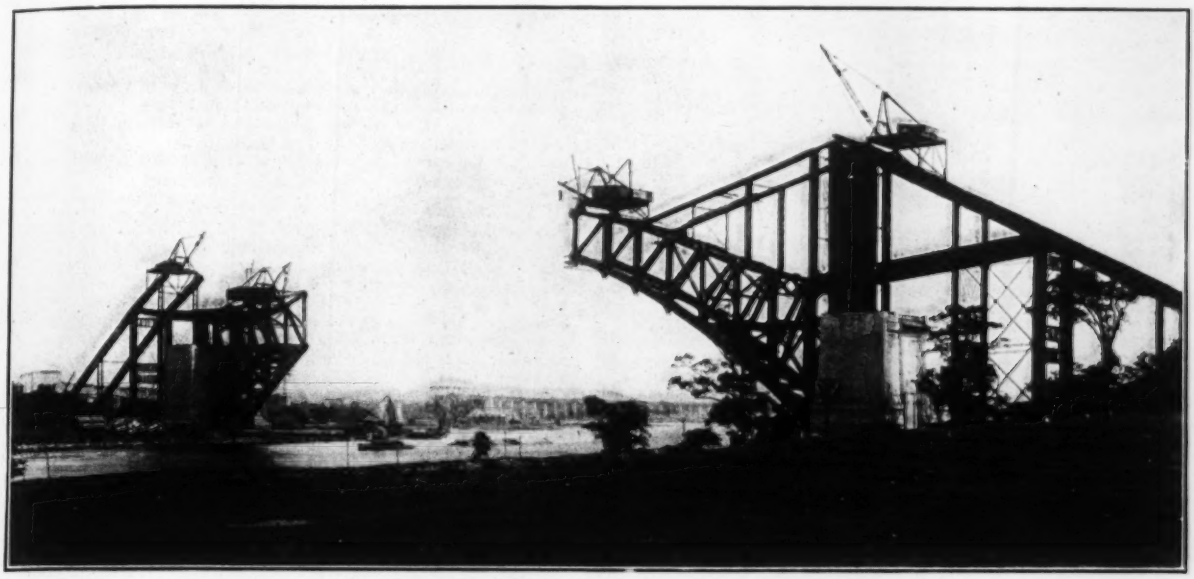


FIG. 2. THE HELL GATE BRIDGE ON JULY 23, 1915

FIG. 3. THE WEST APPROACH OVER WARD'S ISLAND

Cleveland to Have New Railway Terminal Facilities

The Cleveland, Akron & Canton Terminal Railway Co., backed by O. C. Barber, has received a franchise from the Cleveland City Council to build a subway from the lake to the southerly limits of the city, passing under E. 55th St. It is estimated that the proposed tunnel will involve an expenditure of $10,000,000 and will require seven years to construct. The plans contemplate the building of factories along the line of the tunnel, with direct connections to them from the tunnel line. The company has made contracts with the Osborne Engineering Co., of Cleveland, and the Foundation Co., of New York, to prepare plans for the subway, the freight stations, and the lake-front development at the foot of E. 55th St.

The City Council has also granted to the Cleveland & Youngstown Railway Co. a franchise to create a great freight terminal in the lower Broadway district. Some forty city streets will have to be vacated to make the terminal a possibility. The franchise ordinance has now been signed by the mayor, who withheld his approval until the company agreed to an amendment providing for electric traction. The adoption of such traction, however, is conditional upon its use by the railways with which the new line will connect. It is proposed also that the company should be required to build a union station on the Mall.

Cutoff at Camden Bend on the Missouri River

Near Camden, Mo., about 30 miles east of Kansas City, the Missouri River makes a long horseshoe bend. The length of the river channel around this bend is about 11½ miles, and the distance across the neck of the bend is only about 2 miles. During the recent high water in the Missouri, the river spread over the bottom lands and cut across the neck of the bend a channel through which practically the entire flow of the river is now passing. The average fall of the river is about 0.8 ft. per mile. The fall in the new channel across the neck of the bend is stated to be about 8 ft. This steep slope gives the water flowing through the cutoff a high velocity, and it is impossible for boats ascending the river to breast the rapid current.

A steamer belonging to the Kansas City & Missouri River Navigation Co., which was returning from St. Louis towing a barge, attempted to navigate the old channel around the bend. When it arrived at the upper end of the cutoff the strength of the current passing into the cutoff tore the barge free from the steamer, and it was carried down through the cutoff. The steamer turned back downstream and finally overtook the barge 22 miles below the point where it had broken loose.

About $80,000 worth of bank revetment work has just been completed by the government engineers along the Camden bend; and should the river maintain permanently its new location, this work will be useless. It is stated that at one time the land owners in the vicinity offered to obtain the right-of-way and present it to the Government if it would excavate a channel across the bend which the river has now made for itself. This is in accord with the well-known belief among laymen in the Missouri and Mississippi valleys that the way to regulate these great rivers is to cut off all the bends and make a straight course for the stream. The government engineers, of course, refused the offer, pointing out to those that made it that such a cutoff would undoubtedly cause destructive changes in the channel of the river above and below, to say nothing of the injurious effects upon navigation.

Track Subsidence on the Erie R.R. at Dunmore, Penn., caused three freight cars to sink 12 ft. into a coal-mine drift of the Pennsylvania Coal Co. The zone of settlement included three tracks of the line for a distance of about 100 ft., the depth being 12 ft. throughout this length. The hole was filled with ashes.

The Derailment of an Electric Car on the International Ry. at Queenstown, Ont., July 7, which caused the death of 15 persons, has been investigated by a coroner's jury. According to the testimony of the motorman of the derailed car, the usual speed in passing over the sharp curve where the derailment occurred was from three to four miles an hour. He estimated the speed as about 30 mi. an hour when the derailment occurred. The brakes worked all right, but the wheels skidded. He did not know whether there was sand in the sand boxes or not, said it was not customary to use sand in warm weather, and in any event the sand boxes were so located that they would not discharge sand on the rail at a sharp curve. When the motorman realized that the wheels were skidding, he released the air brake and reversed the motor, trying to stop the car by that means. But the trolley had come off the overhead wire and the attempt was ineffectual. Bearing on the overloading of the car at the time of the accident, the conductor said he had taken up 157 tickets before the car went off the tracks.

One of the Worst Marine Disasters ever recorded occurred in Chicago on Saturday morning, July 24, when the excursion steamer "Eastland" capsized while lying in the Chicago River alongside a wharf. About 2500 passengers had gone on board the boat, most of them employees of the Western Electric Co., for a day's excursion. The boat gradually listed over away from the wharf, finally snapped the hawsers securing her, and turned over on her side, precipitating the passengers, a large proportion of whom were on the upper deck, into the river. Up to 11 p.m. on July 25, 826 bodies had been recovered and the total number of missing, reported and estimated, was 691. Counting all the missing as lost, this makes the total casualties 1517. The "Eastland" was a steel hull lake vessel built in 1903, 265 ft. long, 38 ft. beam, with a draft of 23 ft. and a net tonnage of 1218. It was used for several years in excursion business from Chicago to South Haven, Mich., and was for five years in excursion service at Cleveland. Reports state that the vessel, which was built with fine lines for high speed, had been criticized by lake shipping men for its lack of stability, and that extensive alterations were made on the boat last winter to remedy this defect, the upper deck and staterooms being removed. It had been in service this season from Chicago across Lake Michigan to St. Joseph, Mich. Investigations of the accident are in progress by the Department of Commerce and Labor, under the personal direction of Secretary Redfield, by a coroner's jury, and by a federal grand jury.

A Building Lot Sale by the Government in the new town of Anchorage, the southwestern terminus of the Government railroad now under construction in Alaska, netted $133,000. The lots averaged 60x140 ft. and the price realized was about $300 per lot, or $1600 per acre. Three months ago a dozen pioneers constituted the population of Anchorage. On June 1 its population was estimated at 2500.

Reduction of the Engineering Staff of the Pennsylvania State Water Supply Commission has been made necessary by the failure of the legislature to pass an appropriation bill for $60,000 to continue the work of the commission's inventory department. The 1913 legislature appropriated $50,000 for the making of an inventory of the state's water-power resources. The sum of $60,000 was asked for from the 1915 legislature, but this was cut by the legislature to $15,000 and by the governor to $7500. Only two employees, and those are clerks, are to be retained. The engineer employees, 13 in number, including Farley Gannett, chief engineer, have been allowed to resign.

Sewage Disposal at Detroit and elsewhere along the Detroit River is being studied by H. C. McRae for the United States Public Health Service, acting under the direction of Prof. Earl B. Phelps in behalf of the International Joint Commission. In collaboration with Mr. McRae, studies of the sewage-disposal problem of Detroit are being made by Clarence W. Hubbell, Consulting Engineer for the Department of Public Works on Sewerage Investigations. Mr. Hubbell is considering various possible sewage-treatment schemes for Detroit, which may be summarized as follows: (1) Numerous small plants at some of the 53 sewer outlets into the river; (2) a smaller number of plants; (3) one plant for the whole city. The studies are taking into account typhoid statistics and other related data.

A **Union Station for Kansas City, Kan.** (across the river from Kansas City, Mo.) is proposed under a 200-year franchise for the Kansas City Terminal Railway Co., of Kansas City, Mo. Whether there shall be one or two union stations (the latter providing for two groups of roads) is to be decided by public vote. The franchise provides also for a four-track elevated structure from Armourdale to the union station, with a double-track extension to the Rock Island Lines. This is to keep the main-line traffic clear of the switching and industrial traffic in the West Bottoms. Three viaducts across the tracks are provided for, and if the Metropolitan Street Railway Co. builds tracks across these it is to pay either 50 or 66% of the cost (this proposition has not yet been agreed upon). Other provisions include a union ticket office in the city, a freight station handling freight independently of the station on the Missouri side, and agreements as to switching changes. If the franchise is accepted by the city commissioners it will be submitted to public vote at a special election.

A Reform in Federal Building Construction is inaugurated in an order issued by Secretary McAdoo on June 29 to the Supervising Architect of the Treasury Department. Heretofore the amount spent in building a post office or custom house has depended much more upon the energy and political strength of the Congressman from the district in which the building was to be located than upon the needs of the Government. The result has been the erection of costly and elaborate stone buildings in country villages, while localities where improved facilities were greatly needed received inadequate buildings or none at all. It is said that many post-office buildings have been erected at a cost of $50,000 to $150,000 in towns where the total postal receipts would not pay the cost of maintaining the building after completion. Under the order recently issued post-office buildings are divided into four classes—A, B, C and D. Class A buildings may be erected in places where annual postal receipts are over $800,000. Such buildings are to have marble or granite facings and to be fireproofed throughout, and when the locations for the buildings justify it they are to have elaborate architectural decorations. Class B buildings may be erected in cities having postal receipts of from $60,000 to $800,000; they are likewise to be fireproofed throughout, but to have limestone or sandstone facing and less expensive interior finish and decoration. Class C buildings are to be erected in towns having postal receipts of from $15,000 to $60,000, and are to be of brick with stone or terra-cotta trimmings. The floors are to be fireproof, but the roof not necessarily so. Finish and decoration are to be more inexpensive than for class B buildings. Class D buildings are designed for towns having less than $15,000 annual postal receipts. Such buildings need have only the first floor fireproofed, and should be structures which business men would consider a reasonable investment. Plans of buildings hitherto constructed are to be used in designing if they are consistent with the foregoing requirements. Standard specifications and standard details are to be prepared to economize in architectural and engineering work in connection with such buildings. The policy outlined has already been followed by the Treasury Department in the purchase of post-office sites. Of 303 sites for Federal buildings authorized Mar. 4, 1913, only 136 have so far been acquired, and the department has limited its purchases to towns where construction of a federal building is justified.

PERSONALS

Messrs. John A. O'Connor, Assoc. M. Am. Soc. C. E., and Arnold G. Chapman, Assoc. M. Am. Soc. C. E., of Albany, N. Y., have been appointed Executive Engineers of the Flood Abatement Commission of Olean, N. Y.

Mr. Edward E. Krauss, who recently retired from the position of Secretary of the American Concrete Institute, has become District Sales Manager, at Philadelphia, for the Penn-Allen Cement Co., of Allentown, Penn.

Mr. John A. Ferguson, Assoc. M. Am. Soc. C. E., recently Engineer of the Bureau of Building Inspection, Department of Public Safety, Pittsburgh, Penn., has been appointed Engineer and Secretary of the Pittsburgh Building Code Commission.

Mr. Prevost Hubbard, Assoc. Am. Soc. C. E., formerly Chemical Engineer in Charge of the Division of Roads and Pavements of the Institute of Industrial Research, Washington, D. C., has been appointed Chief of the Division of Road Material Tests and Research of the United States Office of Public Roads and Rural Engineering.

Mr. H. L. Beatty has been elected President and Mr. A. O. Beatty, Vice-President of M. Beatty & Sons, Ltd., Welland, Ont. Mr. Harris T. Dunbar, of Buffalo, has been elected a member of the Board of Directors. Messrs. V. R. Browning and B. F. Miles, Directors, and R. A. Greene, M. Am. Soc. M. E., General Manager, have resigned from the company.

Mr. W. E. Moore, Fellow Am. Inst. E. E., Vice-President and General Manager of the West Penn Traction Co., Pittsburgh, Penn., has resigned in order to devote his entire time to consulting engineering practice covering power plants and transmission, car shops and railway engineering; he will also undertake appraisals, property examinations for financing purposes, problems arising from public service regulations and advisory work in connection with public utility management.

Messrs. R. F. Wagner and Carl Monniche have formed a partnership under the firm name of Wagner & Monniche, Designing and Construction Engineers, with offices in the Krise Building, Lynchburg, Va. They will specialize in structural work in both steel and reinforced concrete. Mr. Monniche was for several years Assistant Engineer with the Isthmian Canal Commission on the design of the Balboa and Cristobal terminals. Mr. Wagner was also with the Isthmian Canal Commission in the Division of Fortifications.

OBITUARY

William B. Taylor, a civil engineer employed in the New York Sewer Department, committed suicide July 22.

Robert E. Bustard, Superintendent of Roads of Passaic County, New Jersey, died at St. Luke's Hospital, New York City, on July 19 in his 54th year.

Sir Sanford Fleming, Hon. M. Can. Soc. C. E., one of the oldest and most famous civil engineers of Canada, died July 22, in Halifax, N. S., aged 88 years. From 1871 to 1880 he was Chief Engineer of the Canadian Pacific Ry.

C. Lee French, Superintendent of the Cumberland division of the Baltimore & Ohio R.R., died at Atlantic City on July 18, 48 years of age. Mr. French had been suffering from anemia, and after undergoing a series of blood transfusion operations went to Atlantic City two weeks ago in an endeavor to fully regain his health.

Joseph Strawhorn, Superintendent of the St. Paul (Minn.) Union Depot Co., died at the Bethesda Hospital in that city on July 12, 51 years of age. Mr. Strawhorn was appointed Superintendent in 1903. Later he was Superintendent of the St. Paul Bridge & Terminal Co., and was reappointed Superintendent of the Union Depot Co. in 1912.

ENGINEERING SOCIETIES

COMING MEETINGS

AMERICAN INSTITUTE OF CHEMICAL ENGINEERS.
Aug. 16-18 and 25-28. Annual meeting in Los Angeles, Calif., Aug. 16-18; in San Francisco, Aug. 25-28. Secy., J. C. Olsen, Cooper Union, New York City.

INTERNATIONAL ASSOCIATION OF MUNICIPAL ELECTRICIANS.
Aug. 24-27. Annual convention in Cincinnati. Secy., Clarence R. George, Houston, Tex.

NEW ENGLAND WATER WORKS ASSOCIATION.
Sept. 7-9. Convention in New York City. Secy., Willard Kent, Narragansett Pier, R. I.

TRAVELING ENGINEERS' ASSOCIATION.
Sept. 7-10. Annual meeting in Chicago. Secy., W. O. Thompson, N. Y. C. R.R., East Buffalo, N. Y.

NORTHWESTERN ROAD CONGRESS
Oct. 4-7. At Cedar Rapids, Iowa. Secy., Jas. P. Keenan, Milwaukee, Wis.

Illuminating Engineering Society—At the ninth annual convention of the Illuminating Engineering Society, which is to be held in Washington, D. C., at the New Willard Hotel, Sept. 20-23, a session will be devoted to the subject of street lighting. At this session will be discussed gas lighting, arc lamps, new types of incandescent lamps, and ornamental street lighting. There will be ten sessions in all, with inspec-

tion trips and a reception and banquet among the entertainment features. The headquarters of the society are at 29 West 39th St., New York City.

Appliances and Materials

Equipment for Sand Photomicrography

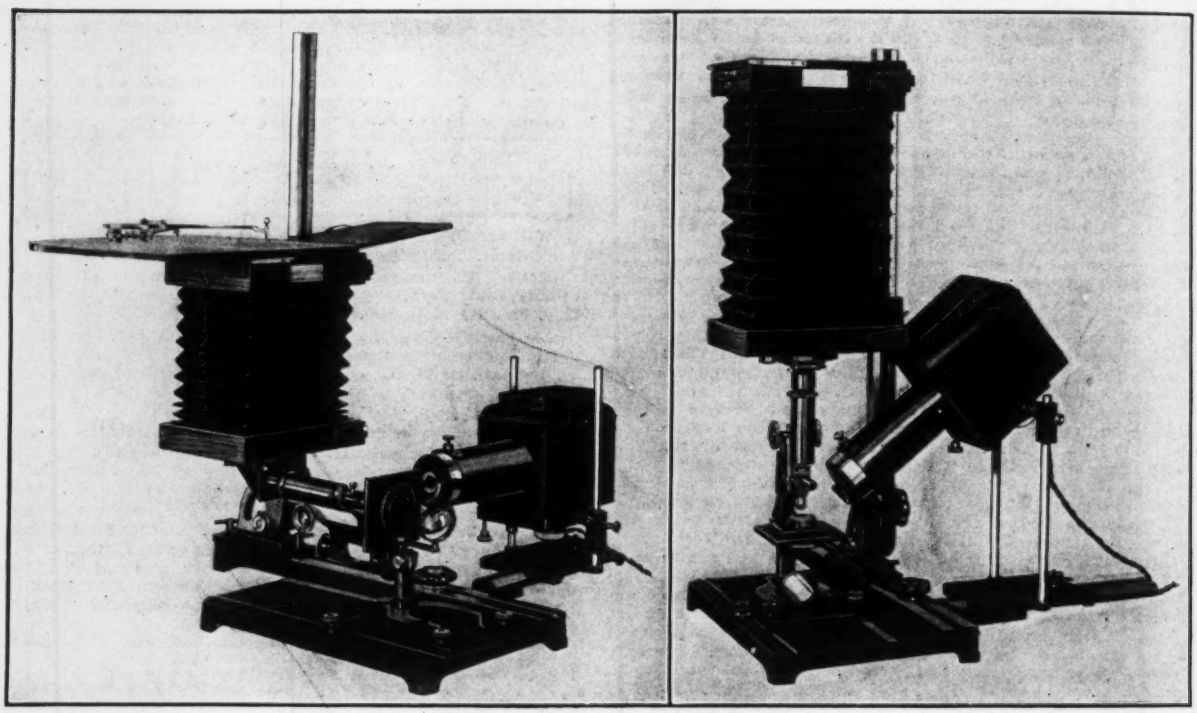
A summary of a paper by N. C. Johnson, entitled "Microstructure of Concrete" before the American Society for Testing Materials was given in "Engineering News," July 8, 1915. The photomicrographic apparatus used in Mr. Johnson's investigations was designed by the Bausch & Lamb Optical Co., of Rochester, N. Y., and is now generally available for concrete testing and investigation.

The apparatus, as illustrated, is assembled on a heavy cast-iron base, one side of which is extended to form a 25-in. milled bed, along which the optical bed carrying the camera may be adjusted. This 25-in. bed is graduated to 60 cm., and has a joint by which it may be inclined and clamped in any position between the horizontal and vertical. The camera-bellows frames have supports which are adjustable along the optical bed and for centering the camera with the microscope. A plate that slides along the base carries clamps to hold the microscope and has two adjustable lugs that permit the removal and return of the microscope without recentering.

The microscope, as illustrated, may be used in either a horizontal or vertical position, the former bringing the ground glass of the camera to a more convenient height for focusing and permitting the full length of the bellows to be utilized. Illumination is furnished by a 250-watt concentrated-filament tungsten lamp and reflector in a housing on two vertical rods projecting from a plate which is adjustable along an optical bed. For oblique illumination of the specimen the bed pivots about the camera-base lock and the lamp-house slides along the vertical rods and tilts on its blocks. The lamp-house, with base and supports, may be removed from the optical bed and used for visual work with the microscope alone. In this case the light is directed obliquely upon the object or from a vertical illuminator. When transparent objects are being examined, the light may be trained upon the mirror under the microscope stage. Photomicrographs of thin sections may be made in similar fashion. At the front of the lamp-house is an extension tube carrying a system of condensing lenses, an iris diaphragm for controlling the size of the beam of light and holders for a ground glass [illegible] ray filter.

For the estimation of the proportions of sand, water, cement and stone entering into a specimen of concrete the planimeter is essential, and an attachment, shown in the view at the left, accommodates the instrument. The attachment is substituted for the focusing screen of the camera and consists simply of a circular ground glass set into a wooden slab. Upon this table runs the planimeter by which the areas of the ingredients, projected upon the ground glass by the microscope, are integrated. The proportion by volume is readily calculated from the areas thus obtained.

Accessories which may be obtained with this outfit comprise a mechanical stage for the systematic examination of the specimen, a specimen mounter, a planimeter attachment, and a lens for the photography of large specimens with the camera alone.



APPARATUS FOR PHOTOMICROGRAPHY OF SAND AND CONCRETE

* * *

Heat Insulation Bricks

A new light nonconductor brick is being made for boiler and furnace settings, breechings, stocks, kilns, ovens, etc., by the Armstrong Cork Co., of Pittsburgh, under the trade name "Nonpareil." Similar insulating bricks have been made in Europe for several years. In making these bricks a mixture of diatomaceous earth, clay and ground cork is molded, dried and fired. The cork is entirely burned out. Each brick is 9x4½x2½ in. and weighs 1½ lb. The conductivity is reported to be 0.5 (B.t.u. per sq.ft. per in. per deg. per hr.), compared with about 5 for firebrick. The material is nearly pure silica and will withstand relatively high temperatures, but it is not intended as a refractory lining. It is recommended as a backing for firebrick, etc. The bricks are laid in a special cement composed of diatomaceous earth and binder. The cost of these bricks is about $36 per M.

* * *

New Steel Pole

A new design of latticed steel pole for electric transmission or distribution lines has recently been marketed by the Carbo Steel Post Co., of Chicago. It is formed by two two-angle flanges riveted over the ends of diagonal lattice bars. The poles are intended to be buried about 4 ft., and below ground are double diagonal lattice bars and a heavy horizontal distance piece. Each flange rests on a triangular foot plate, to each corner of which it is connected by an angle brace.

High-carbon steel is used. It is claimed that the first cost of these poles is about equal to wood but that only half as many are needed on most lines.

Engineering News

VOLUME 74 AUGUST 5, 1915 NUMBER 6

An Ornamental and Cheap Concrete Bridge

By R. C. Hardman*

A reinforced-concrete viaduct that presents some more or less unusual features has recently been completed at Cañon City, Colo. Among these features, aside from the smallness of the city for which the work was done, was the low cost of the structure—approximately $1.40 per sq.ft. of roadway and walk.

Cañon City, having a population of 5162 in 1910, is situated in the center of a large apple-growing territory along the Arkansas River at the mouth of the famous Royal Gorge of the Arkansas. On the south bank of the river is South Cañon, Prospect Heights and Lincoln Park, with a tributary population of some 2000 persons. The river is spanned by three steel highway bridges. In an attempt to draw traffic, and incidentally business, from the tributary population to the old portion of the city, the taxpayers organized an Improvement District and voted bonds for the construction of the viaduct. Thus the structure was built for business reasons rather than from necessity.

FIG. 1. REINFORCED-CONCRETE VIADUCT AT CANON CITY, COLO.
North end is to the left

The viaduct extends over an old swampy river bed, an alley, a large power and irrigation canal, the main line of the Denver & Rio Grande R.R., a small unused irrigation ditch, a roadway, the main line and three sidings of the Atchison, Topeka & Santa Fe Ry., joining a steel river bridge at the south end.

The structure has a roadway 23 ft. 11 in. in width between curbs and one sidewalk 4 ft. 11 in. in width. A vertical clearance of 26 ft. is provided over the Denver & Rio Grande and 23 ft. over the tracks of the Atchison, Topeka & Santa Fe. The north end, 27 ft. of which is an earth fill between retaining walls, has an ascending grade of 5.94% for a distance of 262.67 ft. Over the Denver & Rio Grande the grade is level for a distance of 65 ft., followed by 208.83 ft. of a 1.39% descending grade, which in turn connects with the river bridge by a descending grade of 8.11%, 255.5 ft. in length. The river bridge, a Pratt truss 105 ft. long, was raised 5 ft. at the north end in order to reduce the gradient of the connecting viaduct. The last 50 ft. of the 8.11% grade is an earth fill between retaining walls, from which an earth descent on a 10% grade affords access to the level of the tracks and roadway.

The total length of the viaduct is 742 ft., of which 665 ft. is of concrete and 77 ft. is earth fill. The structure is of a girder-and-slab type, which was selected in order to secure the required clearance over the railroad tracks for the full width of the rights-of-way without entailing the steeper approach grades which an arch type would have necessitated. No uniform length of span was feasible because of the fixed points at which so many of the piers were best located. The spans vary in length from 27 to 44 ft. for the regular spans, with one of 60 ft. and one of 65 ft. over the railroad tracks.

The committee having charge of the building of the viaduct desired it to have as massive an appearance as possible compatible with low cost. For this reason the type of support shown in Fig. 2 was selected instead of the usual slender, heavily reinforced column. As will be seen, the support, or pier, consists essentially of three plain concrete posts joined at the top by semicircular arches, also unreinforced. These posts, which are 4x5

*307 Cheyenne Boulevard, Colorado City, Colo.

ft. in plan in four piers and 4x4 ft. in eleven piers, have battered bottoms resting upon a footing carried to an underlying bed of boulders at depths of from 3 to 9 ft. All piers were constructed before any superstructure was placed.

The superstructure is of the girder-and-slab type designed for a loading of 100 lb. per sq.ft. of roadway and walk, or a 15-ton road roller. The railroad rights-of-

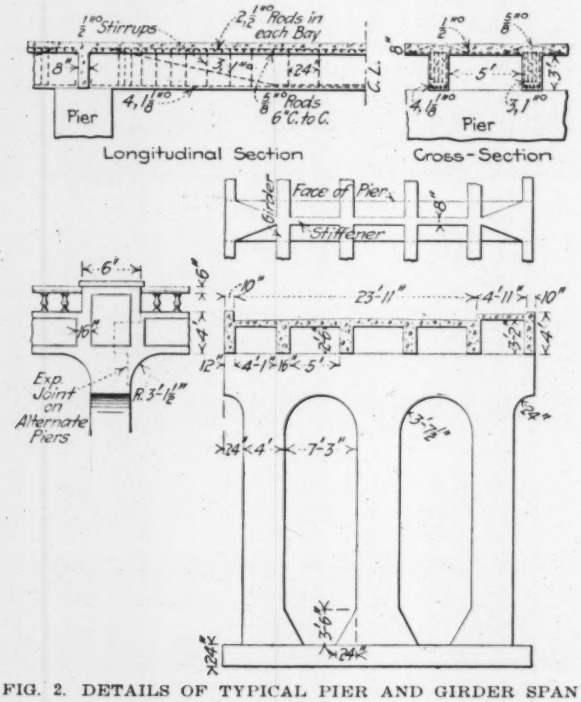


FIG. 2. DETAILS OF TYPICAL PIER AND GIRDER SPAN OF CANON CITY VIADUCT

way are spanned by steel plate-girders carrying a concrete floor-slab as shown in section in Fig. 3, in which a unique lateral stiffening was carried out. The outside girders in each span were incased in concrete on one side only, to conform in architectural treatment to the remainder of the structure. The method by which this was accomplished is clearly shown in Fig. 3.

Typical sections of the concrete spans, which present nothing unusual, are shown in Fig. 2. The placing of the girders on the piers is unique in that all piers are essentially free except for the meager bond between old

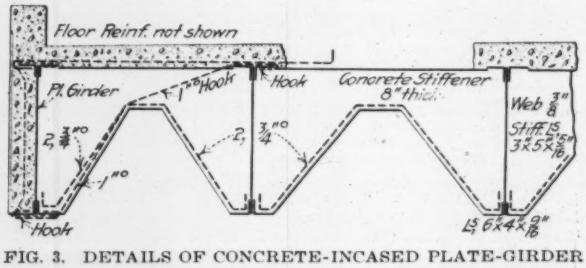


FIG. 3. DETAILS OF CONCRETE-INCASED PLATE-GIRDER

and new concrete, as no reinforcement was used to tie the pier and girder together.

In the construction of forms no timber heavier than a 2x4 was used for the piers, although continuous pouring of concrete to a height of 31 ft. was accomplished. For shoring doubled 2x4's and 2x6's were used with lateral and diagonal bracing of 1x6's and 1x8's. The forms for the girders were built complete on the ground. They were then hoisted by means of one gin-pole, placed in position on the piers, spaced and braced. Each girder form being entirely self-supporting, the floor joists and lagging was next placed, the shoring-up being left until the end.

As the amount available for the structure was limited, the keynote of the viaduct was simplicity in every detail, with a minimum of costly architectural treatment. Practically the only architectural treatment consisted of a few segmental haunches, paneling of the outside girders and a cast balustrade, neither of which was costly. There being no difficulties of design, the only problem was to obtain a combination of massive appearance with beauty and low cost. The degree of success attained may be judged by the accompanying view of the structure.

The bridge was designed and built by the writer as engineer for the Central Construction Co., of Colorado Springs, Colo.

As the site was examined, plans and details made, estimate prepared and contract let in but ten days' time, the viaduct will not, perhaps, stand as a model of engineering design, as many little refinements could well have been introduced had time permitted. However, as an example of what can be done for a comparatively low cost, it may merit this description.

Load Tests on Brick Piers*

The crushing strength of brick piers of large size is under investigation by the Bureau of Standards in Pittsburgh, Penn. So far more than 22 piers have been tested. As nearly as possible each pier was 30 in. square by 10 ft. high, except two piers which were built 5 ft. high. Each pier averaged 44 courses. The joints were about $\frac{5}{16}$ in. thick. The qualities of brick used, kinds of mortar, ages of piers and results of tests are tabulated in Fig. 1. Three typical styles of bonding were selected for test—header every other course, every fourth course and every seventh course; thus there were three types of bonding for each set of piers. One of the 5-ft. piers had steel-mesh reinforcing placed in every horizontal joint; the other, every four courses; but the results obtained were inconclusive (see Conclusion No. 7).

Method of Test

Each pier was constructed in the laboratory on a steel plate 1 in. thick, leveled on the floor. The plate surface which was to rest on the base of the testing machine was planed. A plate $1\frac{1}{2}$ in. thick, with eye-bolts for crane connections, was laid on the top of the finished pier, connected with the baseplate by four vertical rods, so that the piers could be lifted easily and set in the testing machine without injury. The top plate was removed after the pier was in place. The pier was leveled, the top covered with a thin coat of plaster of paris, and the head of the machine brought down, squeezing out superfluous material. The pier was then left for several hours to

*Abstract of paper by J. H. Griffith and J. G. Bragg, Bureau of Standards, Pittsburgh, Penn., presented before the recent meeting of the National Brick Manufacturers' Association. It is based on an investigation being conducted by the Bureau of Standards in Pittsburgh to determine the physical properties of large piers when tested to destruction. The investigation was inaugurated by the N. B. M. A. under its Committee on Technical Investigations (Prof. Edward Orton, Chairman) and was taken up in coöperation with the Bureau of Standards through Prof. A. V. Bleininger, Ceramic Chemist, who acted in an advisory capacity in the conduct of the work.

allow the plaster to set. Measurements of compression were made at several corners, as were also transverse measurements and strains between bricks.

As shown in Fig. 1, bricks laid up in cement mortar stood up best. Cement-lime mortar gave almost as good results. Piers using lime mortar were only about one-half as strong as those with cement; but it should be noted that vitrified brick laid up in lime mortar withstood a pressure of more than 1400 lb. per sq.in. The weakness of these piers was caused mainly by an almost entire lack of carbonation of the mortar on the interior, rather than by any special inefficiency of lime mortar when properly aged.

Characteristic Failure of Piers

At about three-fourths maximum load vertical hair-cracks began to appear. With continued loading the piers broke up into vertical laminations. This condition was induced by the bending of the individual bricks, causing them to split transversely. The bending is the result of nonuniform bearing proceeding from lack of homogeneity of texture in the pier, in an individual brick or in the mortars.

The progressive action of this cracking is most interesting. Sometimes very early in the test small haircracks formed in the upper or lower surface of a brick, usually just under or near the vertical joints, and widened at the surface as the load increased. Finally, subdued sounds

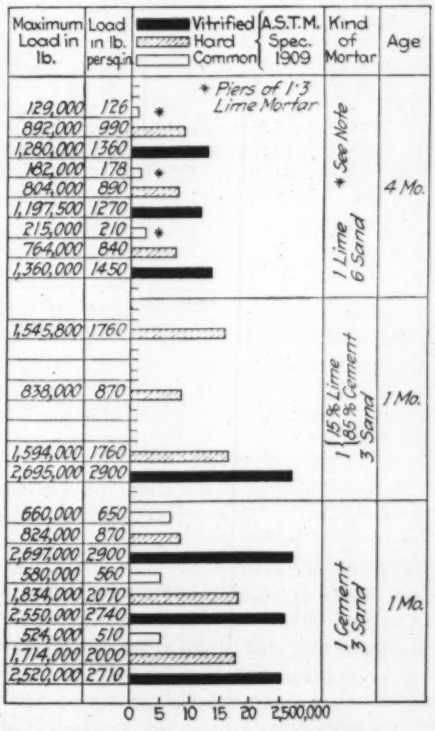


FIG. 1. DATA OF LOAD TESTS

were heard, which increased in volume until, in the case of vitrified and hard-burned brick laid up in lime mortar, a succession of popping sounds occurred, frequently increasing until the sound became a loud "bubbling" in the final stages. When a brick split, the ones above and below tended to do the same, until the structure became a succession of strips, and failed either by spalling out in large masses or as a group of thin, isolated columns.

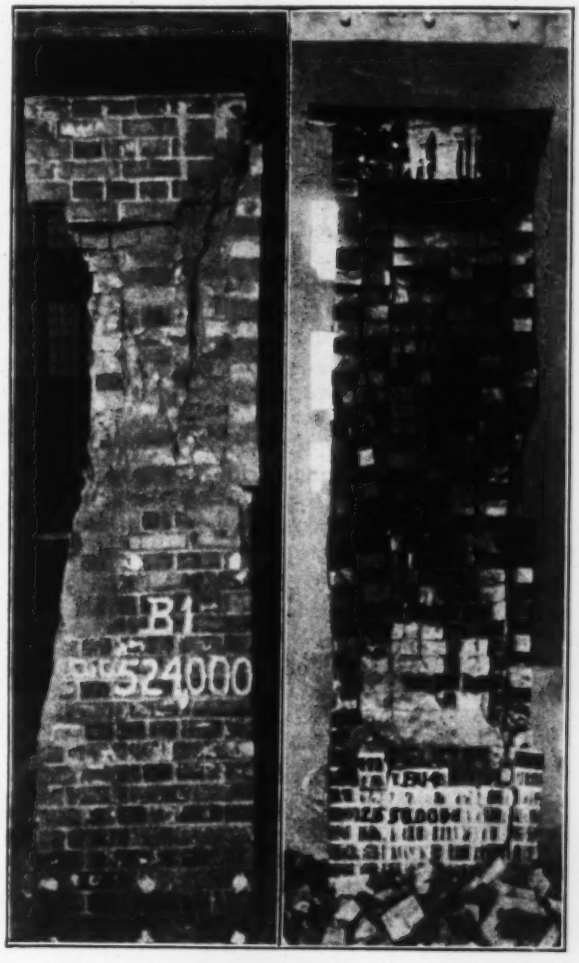


FIG. 2. FAILURE BY SPALLING (LEFT) AND BY VERTICAL CLEAVAGE (RIGHT)

Left—Common brick—cement mortar 1 : 3; maximum unit load, 580 lb. per sq.in. Right—Vitrified brick—cement mortar 1: 3; maximum unit load, 2833 lb. per sq.in.

The following are the main conclusions reached in the investigation:

(1) The strength of piers under vertical loading bears a close relation to the kind of brick used and the quality and age of the mortar.

(2) The low strength of piers laid in lime mortar is attributed in the main to insufficient aging, with a consequent lack of a proper carbonation of the mortar.

(3) Experiments seem to indicate that the strength of a pier is largely independent of the course bonding. The real function of bonds is to maintain a certain integrity and monolithic action of the masonry against initial strains induced through setting and "drying out" of the mortar rather than to give any great increase of vertical strength.

(4) The elastic limit of the piers laid up in cement and cement-lime mortar was approximately one-half the maximum load.

(5) Piers fail through a tendency to separate into vertical strips caused by bending failure in the individual brick produced by unequal distribution of the vertical load over the cross-section. This is aggravated by ineffective shearing and adhesive strength of the mortar.

(6) A high modulus of rupture of the brick is more desirable than superior crushing strength. Higher modulus of rupture would be realized in practice by increasing the thickness of the bricks or by laying them on edge.

(7) A further study of the action of steel-mesh reinforcing in the horizontal joints is recommended in order to ascertain the load efficiency of piers having an outer shell of hard brick with an inner core of softer brick. This construction would be used where a high-grade brick is not easily obtainable.

Sixty-Seven Breaks in a Cast-Iron Water Main

By C. E. Davis*

Sixty-seven breaks have occurred since 1893 in the 30-in. cast-iron main known as the Hartwell Ave. line of the Philadelphia water-works. This main is about 2½ miles long and delivers water from the Roxborough high-service standpipe to Chestnut Hill.

Over a large part of its length the pipe line is laid in unimproved country road, and it is nowhere subjected to heavy or frequent traffic. There has been no disturbance from adjacent excavations, nor has there been any settlement or yielding of the ground in which the line is laid. The pipe has a comparatively light cover of earth and is probably quickly affected by temperature changes.

A parallel main to the Hartwell Ave. line, drawing likewise from the Roxborough standpipe and leading to Chestnut Hill, is laid on Rex Ave. The accompanying profiles show the heads under which the two mains are operated. The Hartwell Ave. profile shows the locations of the several breaks.

The Rex Ave. line was laid in 1902 and has given no trouble. The Hartwell Ave. line, as before stated, has broken 67 times since it was laid in 1893. Both mains are operated under the standpipe head indicated on the profile, and under normal conditions this standpipe is directly connected with the discharge from the pumping station. The mains feed substantially the same distribution system at Chestnut Hill and are cross-connected.

The Hartwell Ave. pipe was furnished by the Reading Foundry & Machine Co., which discontinued the manufacture of pipe a number of years ago. The specifications under which it was manufactured are not available, and no record of tests of the pipe is extant. The thickness of the metal in the body of the pipe appears to be about what is called for under present standard specifications for the head under which it is operated.

The direct cause of these numerous breaks appears to be a poor quality of cast iron, as indicated by the following typical analysis, in which special attention is called to the high phosphorus:

Silicon, 1.85; sulphur, 0.097; manganese, 0.361; phosphorus, 1.33; combined carbon, 0.526; gr. carbon, 2.756; total, 3.282.

Some significance should be attached to the fact that in a distance of one thousand feet easterly from the Wissahickon Creek no breaks have occurred. An analysis of the iron in the pipes laid in this section is not available, but an external examination indicates that it is of a better quality than in the sections where the breaks have occurred. Reports from workmen replacing the broken pipes corroborate the conclusion that the iron is inferior in quality. These reports are invariably to the effect that the pipe is easily broken up.

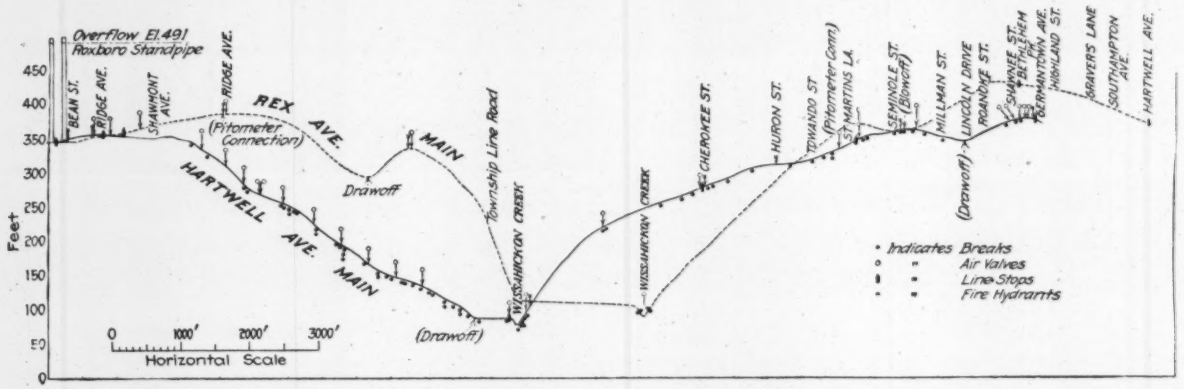
No particular significance can be attached to the character and location in the individual pipes of the several fractures, except that these still further corroborate the conclusion that the metal is poor. Frequently, a piece is found broken out bodily from the barrel of the pipe and entirely freed from the rest of the casting.

The following table gives a record of the 67 breaks by years since the pipe was laid and by months of the year:

BREAKS IN HARTWELL AVE. WATER MAIN

Year	Breaks	Year	Breaks	Month	Breaks
1894	3	1905	4	January	9
1895	5	1906	2	February	8
1896	5	1907	2	March	5
1897	0	1908	3	April	4
1898	2	1909	2	May	4
1899	7	1910	2	June	4
1900	2	1911	4	July	3
1901	6	1912	6	August	4
1902	0	1913	5	September	5
1903	2	1914	3	October	6
1904	1	1915	1	November	8
				December	13
Total for the 22 years			67		

The seasonal variation in the breaks may be significant, but there are no data available which indicate that any difference in the operation of the main could account for the slight variation in the number of breaks in an individual year, these breaks ranging from none to seven in a single calendar year.



PROFILES OF HARTWELL AND REX AVE. WATER MAINS, PHILADELPHIA

The joints on the Hartwell Ave. line are ordinary lead-and-gasket joints. They have given no unusual trouble either from leaks or from blowing out, all the difficulty having been concentrated in broken castings. Air valves are placed on the summits at the standpipe end of the main, and these have been operated regularly for many years. Additional air valves have recently been placed at frequent intervals throughout the entire length of the line. There is no definite record of the actual grades to which the main is laid, but the surface of the ground appears to preclude the probability of summits other than those indicated on the accompanying profile. This conclusion is corroborated by the fact that the additional air valves show no evidence of an accumulation of air in the pipes.

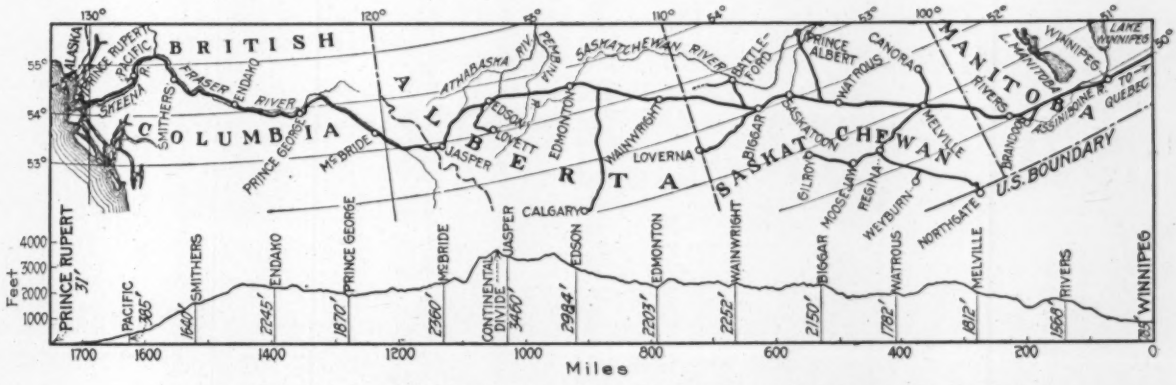
*Chief of Bureau of Water, Philadelphia, Penn.

The absence of air and hammer, the freedom from trouble in the parallel Rex Ave. main, the chemical analysis of the iron from the Hartwell Ave. pipe, and the corroborative testimony of the ease with which the iron is broken by workmen lead to the conclusion that a poor quality of metal was responsible for the frequent breaks in the Hartwell Ave. main.

Grand Trunk Pacific Ry.

The Grand Trunk Pacific Ry. was opened for through commercial traffic between Winnipeg and Prince Rupert, 1748 mi., on Sept. 1, 1914 (steel having been joined Apr. 7, 1914), thus completing the second transcontinental railway in Canada. The eastern section of the line from Moncton, N. B., to Winnipeg, constructed by the Canadian Government, was described in *Engineering News*, June 27, 1912. The western section was built by the Grand Trunk Pacific Ry., and is operated by that company.

Construction was commenced in 1905, and has been under the direction of B. B. Kelliher, Chief Engineer. He resigned his position in the fall of 1914, and the work is now under H. A. Woods, who has been Assistant Chief Engineer for several years. Some of the difficult work along the Fraser River was described in *Engineering News*, Oct. 8, 1914. A condensed map and profile are given in the accompanying illustration. There is a total rise of 2938 ft. from Winnipeg (785 ft.) to the Rocky Mountain summit at Divide (3723 ft.), and a fall of 3705 ft. from Divide to the terminal at Prince Rupert (31 ft.). For operating, the line is divided into 14 divisions, or train districts, and the characteristics of these are shown in the accompanying table.



PLAN AND PROFILE OF THE WESTERN SECTION OF THE GRAND TRUNK PACIFIC RY.

CHARACTERISTICS OF THE GRAND TRUNK PACIFIC RY., CANADA

Train District	Elev. Above Sea Level, ft.	Miles	Ruling Grade, Per Cent. Westbound Per Cent.	Mi.	Eastbound Per Cent.	Mi.	Level, Mi.	Max. Curve, Deg.
1. Winnipeg (785 ft.) to Rivers	1568	142	0.5	17	0.4	1	36	3
2. Rivers to Melville	1812	137	0.5	14	0.4	18	26	3
3. Melville to Watrous	1782	129	0.5	19	0.4	35	10	2
4. Watrous to Biggar	2150	118	0.5	27	0.4	19	15	3
5. Biggar to Wainwright	2252	140	0.5	27	0.4	36	22	3
6. Wainwright to Edmonton	2203	125	0.4	29	0.4	35	20	4
7. Edmonton to Edson	2984	128	0.4	48	0.4	16	13	4
8. Edson to Jasper	3460	104	0.4	38	0.4	26	14	6
9. Jasper to McBride	2360	107	0.4	20	0.4	37	22	6
	....	pusher	...	...	1.0	20	...	...
10. McBride to Prince George	1870	144	0.4	18	0.4	36	32	6
11. Prince George to Endako	2245	116	0.4	24	0.4	7	36	6
12. Endako to Smithers	1640	127	0.4	4	0.4	28	26	6
13. Smithers to Pacific	385	105	0.4	4	0.4	67	15	6
14. Pacific to Prince Rupert	31	120	0.3	1	0.4	11	72	6

The remarkable feature of the profile and the location is the low grade secured through the Rocky Mountains, this being 0.4%, with the exception of a 20-mi. pusher grade of 1%. For the entire line the maximum grade is 0.4%, with an aggregate of a little over 100 mi. of 0.5%. The latter grades are against westbound traffic only, and the heavier traffic will be eastbound. The 1% pusher grade (against eastbound traffic) is on division No. 9. The changing character of the country is indicated by the change in curvature. For the first five divisions from Winnipeg (to Wainwright, 670 mi.) the maximum curves are 3°. For the next two divisions (to Edson, 250 mi.) this is increased to 4°, while on all the remaining seven divisions, 825 mi. to the coast, the maximum curvature is 6°.

The maximum elevation above sea level is 3723 ft. at the Continental Divide, in the Yellowhead Pass. About 30 mi. beyond this, along the Fraser River cañon, comes the sharp fall of the 20-mi. pusher grade. There is also a summit elevation of 3560 ft. at Obed, on the same division. The Canadian Pacific Ry. reaches an elevation of 5330 ft. at Stephen and 4330 ft. in the Rogers Pass, with grades of 2.2% in both cases (originally 4.2%). The Canadian Northern Ry. will use the Yellowhead Pass, and practically parallel the Grand Trunk Pacific Ry. for about 250 mi. from Edmonton.

The railway has been provided mainly with permanent structures, but there are a few wooden bridges of comparatively temporary nature which will be filled from time to time. There also remains the construction of additional stations, section houses and similar facilities as these become necessary with the development of the traffic. Division points, with the exception of that at Prince Rupert, have all been supplied with roundhouses, shops, coal- and water-handling plants, etc. The railway is in good condition for traffic, but (as on all new lines) considerable additional ballasting will be required where settlement occurs. This work will be done from time to time, and a further lift of ballast will be given over the entire line after traffic has thoroughly settled the roadbed.

At Prince Rupert, the Pacific port, temporary facilities have been provided for railway terminals and dockage. A very ambitious layout is contemplated for railway and water terminals, but it has been thought desirable to make extensive studies before commencing such an undertaking, and this preliminary work is still under consideration.

Sea Water to Rise into Fresh-Water Canal

By Paul Whitham*

In *Engineering News*, Nov. 26, 1914, p. 1088, there was an editorial commenting on the salting of Miraflores Lake on the Panama Canal as a "curious hydraulic paradox."

At Seattle, Wash., what is known as the Lake Washington Canal is now approaching completion. The community had always supposed that the water above the locks would be fresh, being fed by the overflow from Lakes Union and Washington, both fresh-water bodies. The unexpected invasion of Miraflores Lake by the sea water led to the belief that a similar condition would prevail at Seattle when the canal and lock are put in operation. The phenomenon is of such interest that it invites an analysis of the reasons therefor and the drawing of a parallel between the two cases.

Briefly, the conditions at Panama are as follows: The Miraflores locks are located about 8 mi. inland from

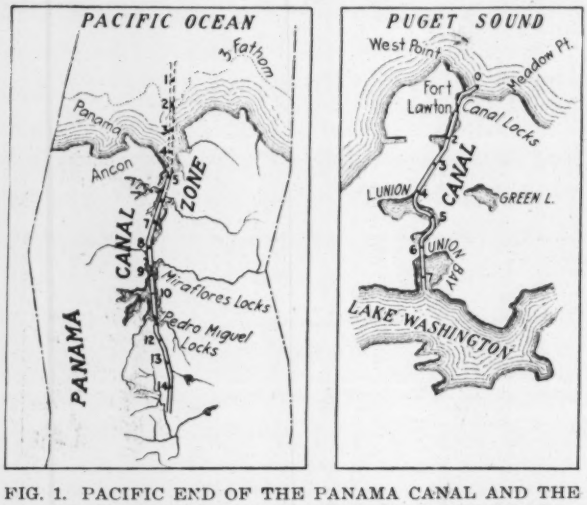


FIG. 1. PACIFIC END OF THE PANAMA CANAL AND THE LAKE WASHINGTON CANAL TO THE SAME SCALE

the Bay of Panama. The extreme tidal range is approximately 20 ft. Miraflores Lake, an irregular body, is about 2 mi. in its greatest length. The surface of the lake is 55 ft. above mean tide in the Pacific, and the bottom of the lake is above high tide in the sea-level section of the canal.

There are two pairs of locks, each 110x1000 ft., dividing the lift into practically two parts. By February, 1914, samples taken showed a diffusion of salt throughout the whole lake, the proportion of salt water running as high as 15 per cent. This necessitated the abandonment of Miraflores Lake as the source of domestic water-supply.

The conditions in Seattle are substantially as follows: The lock is located about a mile inland from the main body of Puget Sound. Salmon Bay, immediately above the lock, is a basin over a mile in length. Lake Union, approximately the same area as Miraflores Lake, begins about 2½ mi. above the lock. A canal connecting the two bodies is nearly 1½ mi. long. The extreme tidal range is approximately 17 ft. The surface of Salmon Bay, which will become a basin or lake, will be about 16½ ft. above mean tide, or only 8 ft. above high tide. The bottom of the basin will be from 24 to 32 ft. below high tide. The lock is 80x825 ft., there being but one lift.

From appearance the conditions here tending to pass sea water through the lock are decidedly more favorable than at Panama. At first thought one might look upon the statement that sea water will pass up the lock as an assertion that the law of gravity will be reversed by making water run uphill. When analyzed, however, the action is shown to be the result of natural laws, explained as follows:

Assume that a vessel enters the Seattle lock from the sea at mean tide. The depth of the water in the lock would be 33½ ft., not less than 80 per cent. of which probably would be salt water. Fresh water from above would then be let in at the bottom of the lock, which action should mix the fresh and salt water during the process of increasing the depth to 50 ft. The resultant mixture would then be approximately 60 per cent. of salt water.

Sea water may be said to weigh 64 lb. per cu.ft. and fresh water, 62.5 lb., a difference of 1.5 lb. The 60 per cent. salt water would consequently weigh 0.90 lb. per cu.ft. more than the fresh water, so that in the lock at a depth of 1 ft. below the surface there would be a head of 0.90 lb. per sq.ft. over that of the fresh water above the lock. On the upper sill, 36 ft. below the surface, the difference in weight would be 32.4 lb. per sq.ft., the equivalent of over 6 in. of head. When the upper gate is opened to let the ship into Salmon Bay this difference in head will be sufficient to cause the brackish water to flow out of the lock with an initial head of 6 in. along the bottom of the basin, while the fresh water will flow in from the top and take its place. When this heavy sea water has spread out and fallen half the distance there will still be a 3-in. head, which should be sufficient to produce a strong current of large volume, considering the resistance encountered in a wide channel. This difference in head operates in principle much the same as a common hot-water heating system, in which a slight difference in head created by the variation of temperature of the water in the up and down pipes is sufficient to produce a positive circulation against the high resistance of small piping.

Unless artificially disturbed, the salt water will tend to spread over the bottom of Salmon Bay and along the canal to Lake Union in wedge-shape form, thus losing head. The movement of boats, the churning of their propellers, and other causes will gradually diffuse the waters of the basins and canal until the entire body becomes brackish. The effect of this stirring up of the water would be mechanically to lift the salt, distributing it all the way up to the surface.

To illustrate: Take a point at some distance above the lock and assume that the 60 per cent. sea water has filled the bottom of the channel to one-fourth the depth, or 9 ft. This would be equivalent to a head of 1½ in. at the bottom of the channel. When the water is thoroughly stirred up, there still would be a 1½-in. head at the bottom, the salt creating the head having, however, a greater elevation—that is, the plane of mean head would be raised from 4½ ft. above the bottom to 18 ft., greatly increasing the volume of water tending to displace the fresh water further up.

*Consulting Civil Engineer, Seattle, Wash.

The effect would be that of having mechanically stored up energy by lifting the weight the additional height, the result being a progressive acceleration of the salt water up the canal and its distribution throughout the basins. The lower temperature of the sea water—the difference between it and the lake water being quite appreciable in the summer—would tend to increase the effective head.

Practically there is a serious side to the salting of Salmon Bay and Lake Union. It is not known how salty the water will become, or whether it will sufficiently approach the condition of sea water to sustain the life of the teredo, or wood borer, that infests Puget Sound waters. In case it did, however, prevent the use of untreated timber piling it would frustrate one of the important benefits expected of the canal lock.

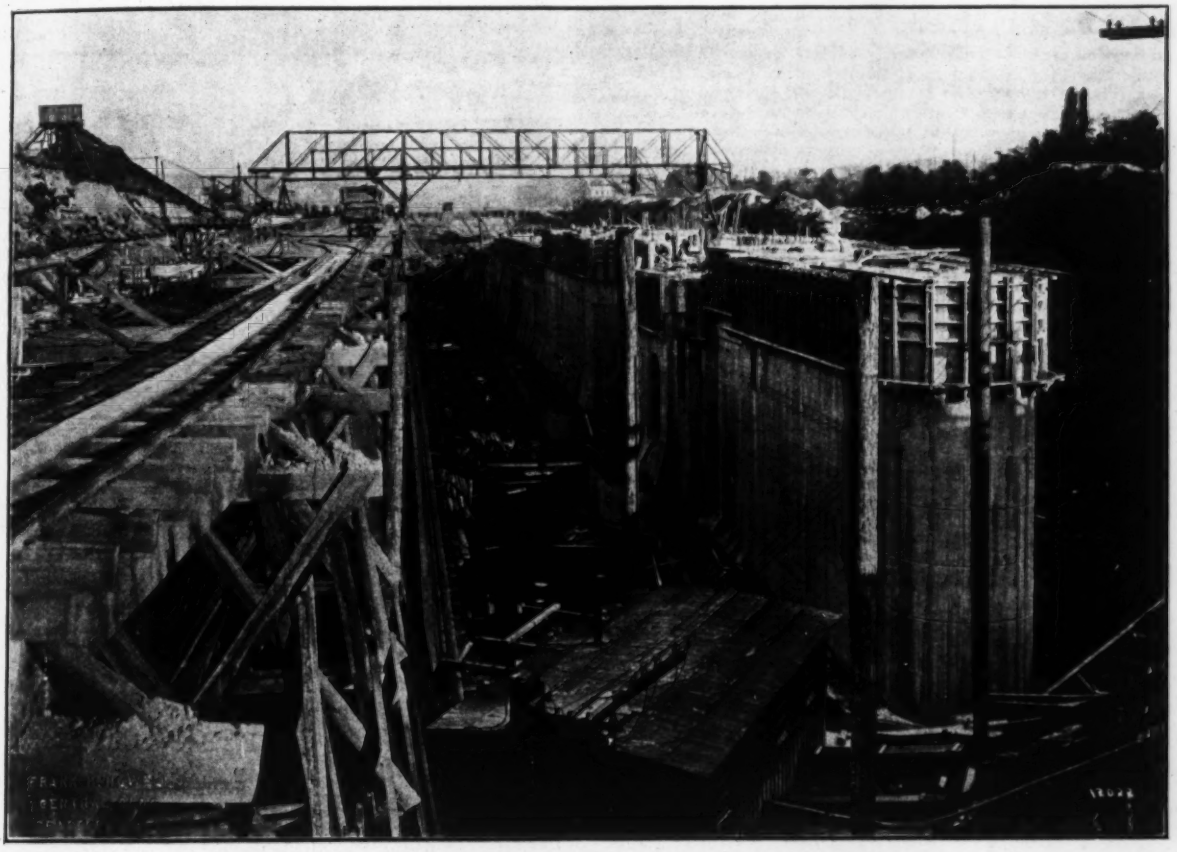


FIG. 2. LAKE WASHINGTON CANAL LOCK UNDER CONSTRUCTION

Industries have already located or purchased sites about Salmon Bay and Lake Union, anticipating the advantages they would enjoy in obtaining fresh water direct from the canal and lakes for manufacturing purposes. The salting of the water will wipe out the expected benefits.

It has been predicted that the fresh-water harbor would become the haven of vessels laid up when out of commission, where they would not be subject to the rapid accumulation of barnacles and other sea growths. Also, in the case of steel vessels, the chemical and galvanic corrosion would be much less in fresh water. These expected advantages, particularly the last-mentioned one, would be endangered by the salting of the water.

J. B. Cavanaugh, Corps of Engineers, U. S. A., is looking into the practicability of preventing or reducing in some manner the passage of salt water through the lock.

Engineers and Public-Service Commissions

In *Engineering News*, Apr. 29 (p. 842), were published suggestions formulated by a committee of engineers representing the national engineering societies for submission to the New York State Constitutional Convention. This general committee of engineers has since organized subcommittees to deal with the various subjects covered in its recommendations. The Subcommittee on Public Utilities has formulated and had introduced into the convention, through the courtesy of Prof. O. H. Landreth, of Schenectady, a member of the convention, a constitutional amendment (No. 708) reading as follows:

Article five of the constitution is hereby amended by adding a new section thereto, to be appropriately numbered, to read as follows:

A public service commission of five members shall be appointed by the governor by and with the advice and consent of the senate. Each commissioner shall hold office for ten years. The terms of the commissioners shall be so arranged that the term of one member shall expire upon the first day of February in alternate years. Each commissioner should have had experience in connection with public utilities which would fit him for the duties of the office.

The commission shall regulate and supervise all common carriers, all water supply, irrigation, drainage, gas, power, lighting, heating, intelligence-transmitting and other public utility corporations operating within the limits of New York state, including similar activities on the part of any other state department or political subdivision of the state.

Removal by the governor of any commissioner should be made only after the filing of charges and after affording the accused an opportunity to be heard in the matter, provided, however, that at any time within the first six months after making an appointment the governor may exercise the power